

FINAL REPORT
A QUANTITATIVE EVALUATION
OF THE
GEOMETRIC ASPECTS OF HIGHWAYS

DEC. 1962

NO. 25

Joint
Highway
Research
Project

by
K. J. THARP

PURDUE UNIVERSITY
LAFAYETTE INDIANA



Final Report

A QUANTITATIVE EVALUATION OF THE GEOMETRIC ASPECTS OF HIGHWAYS

TO: K. B. Woods, Director
Joint Highway Research Project

December 18, 1962

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

File: 8-4-23
Project: C-36-17W

Attached is a Final Report titled "A Quantitative Evaluation of the Geometric Aspects of Highways". It has been prepared by Kenneth J. Tharp of our staff under the direction of Professor M. E. Harr. Mr. Tharp also completed his work for the Ph.D. degree and utilized the research reported in the attached publication as his thesis.

The study reported is an investigation of a quantitative measure of the resistance to the flow of traffic of certain geometric highway features. The mechanistic model considered resulted from the postulate that traffic reacts to a motivating pressure potential which reaction is reflected in the behavior of the traffic as it traverses the section of highway in which the geometric feature is located. The results of the study reveal that the developed model conforms closely to the observed speeds in the vicinity of geometric features except for highway features which require extreme speed changes.

The report is presented for the record.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HIM:kmc

Attachment

Copy:

F. L. Ashbaucher
J. R. Cooper
W. L. Dolch
W. H. Goetz
F. F. Havey
F. S. Hill
G. A. Leonards

J. F. McLaughlin
R. D. Miles
R. E. Mills
M. B. Scott
J. V. Smythe
J. L. Waling
E. J. Yoder

Vol. 10, No. 1

CHICAGO, ILL., JANUARY 1, 1911

Published by the
American Medical Association

Subscription price, Five Dollars per Annum in Advance
Single Copies, Fifteen Cents

Entered as Second-Class Matter, May 26, 1904, under Post Office No. 383, at Chicago, Ill., under special permission of the Post Office and General Land Office. Acceptance for mailing at special rate of postage provided for in Act of October 3, 1917, authorized on July 10, 1918.

Postpaid. This journal is published weekly, except on Sundays and public holidays, when it is published on the following day. It is sent by mail to subscribers at the rate of five dollars per annum in advance. Single copies are sold at fifteen cents. The subscription price includes the journal and the right to use the journal in the library of the subscriber. The journal is not to be used for any other purpose. The journal is not to be used for any other purpose. The journal is not to be used for any other purpose.

Published by the American Medical Association

535 North Dearborn Street, Chicago, Ill.

Telephone: 521-1111

Subscription price, Five Dollars per Annum in Advance

Single Copies, Fifteen Cents

Published by the American Medical Association

535 North Dearborn Street, Chicago, Ill.

Telephone: 521-1111

Subscription price, Five Dollars per Annum in Advance

Single Copies, Fifteen Cents

Published by the American Medical Association

535 North Dearborn Street, Chicago, Ill.

Telephone: 521-1111

Subscription price, Five Dollars per Annum in Advance

Single Copies, Fifteen Cents

Final Report
A QUANTITATIVE EVALUATION
OF THE
GEOMETRIC ASPECTS OF HIGHWAYS

by

Kenneth John Tharp
Research Engineer

Joint Highway Research Project
File No: 8-4-23
Project No: C-36-17W

Purdue University
Lafayette, Indiana
December 18, 1962

ACKNOWLEDGMENTS

The author's presence at Purdue University was made possible by a sabbatical leave from Duke University, Durham, North Carolina in conjunction with an Automotive Safety Foundation Fellowship. He is very appreciative of the opportunity afforded him to undertake this task. In addition to these sources, funds for equipment, materials and labor were provided by the Joint Highway Research Project of Purdue University, Professor K. B. Woods, Director.

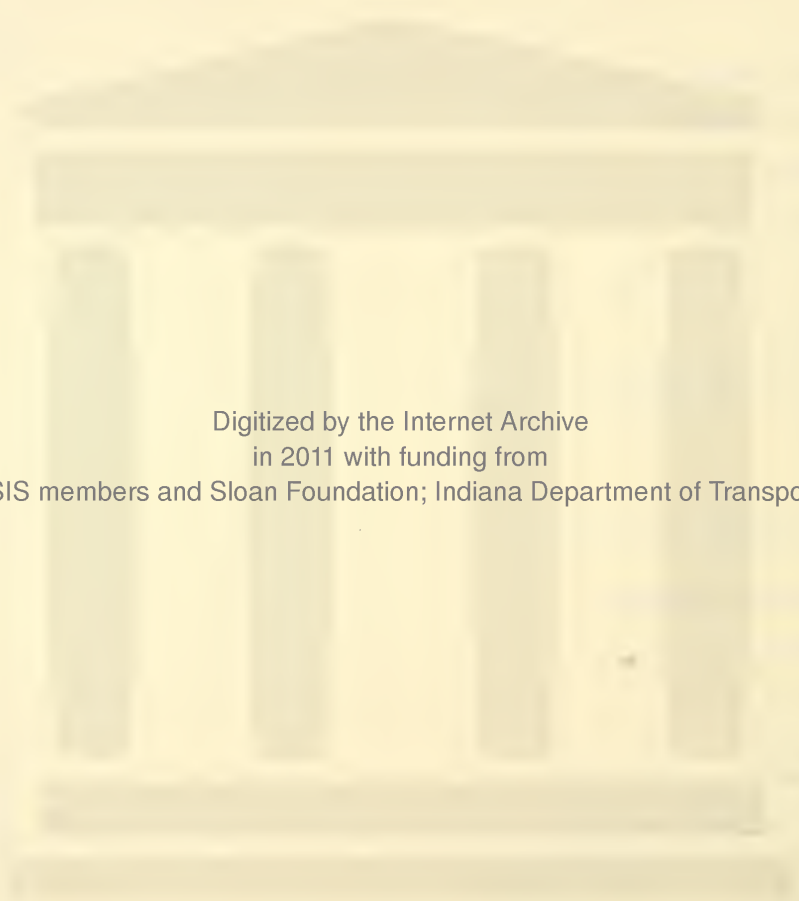
The author is indebted to Drs. M. E. Harr and G. A. Leonards for the original idea of the subject material of this work. Further gratitude is expressed to Dr. Harr for his guidance and direction throughout the research and writing.

Special acknowledgment is due to the author's major professor, H. L. Michael, for his encouragement and support. Gratitude must be expressed to Prof. Jess Senn and the staff of the Audio-Visual department for their cooperation and for making photographic equipment available.

The author wishes to express his gratitude to his children who sacrificed much during the time required for this task. And above all, the author is indebted to his wife for her encouragement, understanding and aid which led to the successful completion of this undertaking.

TABLE OF CONTENTS

	Page
LIST OF TABLES	v
LIST OF FIGURES	viii
ABSTRACT	x
INTRODUCTION	1
REVIEW OF LITERATURE	4
General	4
Theories	5
Statistical Studies	7
Quantitative Evaluation of Features	8
THEORY	10
Assumptions	10
Formulation of Problem	11
Application	11
DEVELOPMENT OF THEORY	13
Approximate Solution	17
PILOT STUDY	20
General	20
Procedure for Pilot Study	20
Data Preparation	25
Data Analysis Procedure	34
Fit of Theoretical Curve	34
Individual Vehicle Statistics	35
Results	36
Fit of Theoretical Curve	36
Open Highway Speed (v_0)	37
Speeds within the Feature (v_1)	37
Speed Changes ($v_0 - v_1$)	37
The Ratio $\frac{v_0 - v_1}{v_0}$	38
The Term e^{-F_0}	38



Digitized by the Internet Archive
in 2011 with funding from

LYRASIS members and Sloan Foundation; Indiana Department of Transportation

TABLE OF CONTENTS (continued)

	Page
Modulus of Geometric Aspects (F_0)	38
Summary and Recommendations	40
EXTENSION OF STUDY	41
Sites	41
Field Procedure	41
Preparation of Data	42
Curves	43
Curve on U. S. 41 (and 52)	43
Right Angle Corner on Indiana 26	46
Transition Sections	49
U. S. 52 4-Lane to 2-Lane	49
Approach to Wm. H. Harrison Bridge	50
Narrow Bridge	54
Indiana 43 near Chalmers	54
CONCLUSIONS	58
RECOMMENDATIONS	59
SELECTED LIST OF REFERENCES	60
APPENDIX I	64
Nomenclature	64
APPENDIX II	65
Appendix Tables	65 - 96
VITA	97

LIST OF TABLES

Table	Page
1 Analysis of Variance for the Significance of the Pressure Potential Theory	34
2 Statistical F Values for the Significance of the Pressure Potential Theory	36

APPENDIX TABLES

A1 Velocity Relationship ($\frac{v(x, t)}{v_{avg}}$) for Selected F_0 Values	65
A2 Error Introduced by Using the Approximate Equation for the Velocity Relationship	66
A3 Vehicle Speed by Stations on the Approach and Through the Curve on U. S. 24 near Reynolds, Indiana	68
A4 Frequency of Speeds of Individual Vehicle Approaching Curve on U. S. 24 near Reynolds, Indiana	70
A5 Frequency of Minimum Speeds of Individual Vehicles on Curve of U. S. 24 near Reynolds, Indiana	71
A6 Frequency of Speed Changes of Individual Vehicles at Curve on U. S. 24 near Reynolds, Indiana	72
A7 Frequency of Ratios of Speed Changes to Approach Speeds for Individual Vehicles for Curve on U. S. 24 near Reynolds, Indiana	73
A8 Frequency of e^{-F_0} Factors for Individual Vehicles Traversing Curve on U. S. 24 near Reynolds, Indiana	74
A9 Frequency of F_0 Values for Individual Vehicles Traversing Curve on U. S. 24 near Reynolds, Indiana	75
A10 Vehicle Speeds by Stations on the Approach To and Through the Curve on U. S. 41 (and 52) near Earl Park, Indiana	76

APPENDIX TABLES (continued)

Table	Page
A11 Frequency of Speeds of Individual Vehicles Approaching Curve on U. S. 41 (and 52) near Earl Park, Indiana	77
A12 Frequency of Minimum Speeds of Individual Vehicles on Curve of U. S. 41 (and 52) near Earl Park, Indiana	78
A13 Frequency of Speed Changes of Individual Vehicles at Curve on U. S. 41 (and 52) near Earl Park, Indiana	79
A14 Frequency of Ratio of Speed Changes to Approach Speeds for Individual Vehicles for Curve on U. S. 41 (and 52) near Earl Park, Indiana	80
A15 Frequency of e^{-F_0} Factors for Individual Vehicles Traversing Curve on U. S. 41 (and 52) near Earl Park, Indiana	81
A16 Frequency of F_0 Values for Individual Vehicles Traversing Curve on U. S. 41 (and 52) near Earl Park, Indiana	82
A17 Vehicle Speeds on the Approach To and Through the Corner of Indiana 26 near Pine Village, Indiana	83
A18 Frequency Distribution of Factors (v_0 , v_1 , $v_0 - v_1$, $\frac{v_0 - v_1}{v_0}$, e^{-F_0} , F_0) for Corner on Indiana 26 near Pine Village, Indiana	84
A19 Vehicle Speed by Stations for the 4-Lane to 2-Lane Transition on U. S. 52 near Templeton, Indiana	86
A20 Frequency of Speeds of Individual Vehicles Approaching Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana	87
A21 Frequency of Minimum Speeds of Individual Vehicles on Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana	88
A22 Frequency of Speed Changes of Individual Vehicles During Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana	89
A23 Frequency of Ratios of Speed Change to Approach Speeds for Individual Vehicles During Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana	90

APPENDIX TABLES (continued)

Table	Page
A24 Frequency of e^{-F_0} Factors for Individual Vehicles During Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana	91
A25 Frequency of F_0 Values of Individual Vehicle During Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana	92
A26 Vehicle Speeds on Ramp and Merging Section of Approach to Wm. Harrison Bridge in West Lafayette, Indiana	93
A27 Frequency Distribution of Factors (v_0 , v_1 , $v_0 - v_1$, $\frac{v_0 - v_1}{v_0}$, e^{-F_0} , F_0) for Merging Section of Approach to Wm. Harrison Bridge in West Lafayette, Indiana	94
A28 Vehicle Speeds by Stations on the Approach to the Narrow Bridge on Indiana 43 near Chalmers, Indiana	96

LIST OF FIGURES

Figure		Page
1	Schematic Illustration of Roadway Section	12
2	Graphical Portrait of Theoretical Speed Ratio Distribution .	13
3	Theoretical Speed Ratio-Distance Relationships for Selected Values of F_0 Above 0.6	15
4	Theoretical Speed Ratio-Distance Relationships for Selected Values of F_0 Below 0.6	16
5	Theoretical Speed-Distance Relationship	17
6	F_0 Graph for Selected Values of v_0 and Δv	19
7	Pilot Study Site	21
8	Data as Recorded by the Photographic Method	24
9	Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana. Data Set Number 1	27
10	Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana. Data Set Number 2	28
11	Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana. Data Set Number 3	29
12	Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana. Data Set Number 4	30
13	Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana. Data Set Number 5	31
14	Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana. Data Set Number 6	32
15	Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana. All Data Sets	33
16	Site of Curve on U. S. 41 (and 52) near Earl Park, Indiana .	44

LIST OF FIGURES (continued)

Figure	Page
17 Speed-Location Graph for Curve on U. S. 41 (and 52) near Earl Park, Indiana	45
18 Site of Corner on Indiana 26 near Pine Village, Indiana . .	47
19 Speed-Location Graphs for Corner on Indiana 26 near Pine Village, Indiana	48
20 Site of 4-Lane to 2-Lane Transition Section on U. S. 52 near Templeton, Indiana	51
21 Speed-Location Graph for Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana	52
22 Site of Merging Section on Approach to Wm. Harrison Bridge in West Lafayette, Indiana	53
23 Speed-Location Graph for Merging Section of Approach to Wm. Harrison Bridge in West Lafayette, Indiana	55
24 Site of Narrow Bridge on Indiana 43 near Chalmers, Indiana .	56

ABSTRACT

Tharp, Kenneth John. Ph. D., Purdue University, January 1963. A Quantitative Evaluation of the Geometric Aspects of Highways. Thesis Professor: Milton E. Harr.

This study is an investigation of a quantitative measure of the resistance to the flow of traffic as offered by geometric highway features. Under consideration is a mechanistic model resulting from the postulate that traffic reacts to a motivating pressure potential which in turn reflects the behavior of the traffic traversing a particular section of highway. When solved, the governing differential equation yields a parameter called the modulus of geometric aspects. This parameter is a measure of the ease with which traffic traverses the given roadway section.

To evaluate the developed model and determine the reasonableness of the modulus of geometric aspects, a detailed study was undertaken of vehicle speeds on an actual highway curve. A procedure was developed whereby the spot speeds could be calculated from observations recorded by photographic means. Statistical methods were used to analyze the data and to determine the goodness of fit of the theoretical and observed speed distributions.

The success of the results obtained from the study of the first highway curve indicated the advisability of extending the study to additional geometric highway features (other curves, merging conditions, etc.). Additional field experiments were conducted to provide a more generalized basis of

evaluating the reliability of the developed modulus.

The results of the study reveal that the mechanistic model as developed conforms closely to the observed speeds in the vicinity of geometric features except for special highway features requiring extreme speed changes. Subject to the same condition, the modulus of geometric aspects provides a reproducible quantitative rating of the geometric highway feature.

INTRODUCTION

Since the sale of the first automobile in 1896, highway transportation has been constantly increasing as a vital factor in the economy of our nation. This expansion necessitates the annual expenditure of billions of dollars for both new construction and the rebuilding of obsolete and worn-out highway and street facilities. In the design of these facilities, the highway and traffic engineer has long been handicapped by only a limited general knowledge of driver behavior within or near the various geometric features constituting the road system.

Present day design of highways generally calls for geometrics based upon some arbitrary standard. As an example the 1956 AASHO policy for the interstate system (1)* lists a desired maximum curvature of 5 degrees and desired gradients not to exceed 4 percent (both for 60 mph design). The use of these standards is justified by observed "satisfactory" performance on existing roadways. However, a designer may logically ask; What variation in driver response will occur with a change from a 5 degree curve to an 8 degree curve? What will happen to traffic flow if 600 feet of 6 percent grade replaces 900 feet of 4 percent grade? What is the effect of combining various curves and grades? These and many other questions can not be adequately answered at the present time and indicate a major deficiency in present traffic flow knowledge and theory.

* Numbers in parenthesis refer to listing in the selected list of reference.

In comparison other fields of engineering have utilized classifications of the subject materials. Generally these classifications are based upon specific properties of the object under consideration. For example, ferrous alloys may be described as having a certain Brinell hardness number, some elastic limit, an ultimate stress, a modulus of elasticity, etc. Knowing these particular values, an engineer can select the proper alloy to meet the specific requirements of a job. As in the other engineering disciplines, it would be extremely desirable if a similar classification could be made to apply to geometric aspects of highway and traffic flow. This classification (or rating) system would then provide the design engineer with the necessary information to enable him to select the most suitable or optimum geometric feature (or features) for any particular section of roadway. In addition, it would be possible to secure a highly desirable "balanced" highway design where all geometric features on a road would produce similar driver response. To achieve this end it is necessary that a rating system be evolved that evaluates all types of geometric aspects on a common basis and without bias.

The development of traffic flow theory has been slow because it is one of the most complex problems facing the engineering profession. Gerlough and Mathewson (2) wrote:

The solution to any problem involving the flow of street and highway traffic should ideally involve consideration of aspects involving many different disciplines. In addition to the obvious physical and psychological problems, full treatment of any situation should include economic, social, political, and public relations, and in some cases, even physiological considerations.

Consequently most mathematical models have to be simplified by restricting assumptions that generally limit the result to a particular site and then under a particular set of ambient conditions. The scores of factors and

and interactions affecting traffic produce a wide variety of human reactions to various roadway stimuli. Driver behavior is generally considered as having a stochastic distribution - that is possessing statistical variations and changing with some suitable parameter (3, 4). For example, vehicle speed studies show free flow speeds clustering around a central value although this value may (and does) vary considerably under different ambient conditions (5, 6, 7). In present day highway design, the general policy is to provide facilities and regulations for the vast majority of drivers - such as the 85th percentile - and to neglect the nonconforming few (8, 9).

The primary objective of this study is to evaluate a specific method of rating geometric features of highways (10). The rating - named the "modulus of geometric aspects" and to be shown symbolically as " F_0 " - is a measure of the ease (as determined by speed changes) with which a vehicle may traverse a section of highway with a particular geometric feature, that is, F_0 is the reciprocal of resistance offered to the vehicle.

Secondary objectives are: (a) To study the velocity changes of vehicles as they approach some particular geometric features: (b) To evaluate photographic methods of data collection which provide for continuous observation of a vehicle on a section of roadway; and (c) To devise methods of analyzing the recorded data.

REVIEW OF LITERATURE

General

As stated by Wardrop (11):

The history of any sciency suggests that progress in any field of research can best be achieved by a judicious mixture of practical experience, experiment and theory.

In traffic engineering each highway, road, and street, beyond its obvious use, can serve a dual purpose - first as an experiment to verify existing knowledge and the beliefs advocated during design and second as an extension of practical experience through the observation of actual traffic operations on the facility. Full advantage of the data available from existing roadways has never been utilized because the lack of a suitable theory around which to evaluate it has lagged.

A roadway is a combination of many fixed and varying factors. Upon the completion of construction, the geometric features of the facility - vertical and horizontal alignment, cross-section configuration, etc. - may be considered as fixed. The ambient conditions under which the vehicles operate - weather, road characteristics, vehicle characteristics, traffic, etc. - vary considerably during an hour, a day, a week, a year, and the lifetime of the roadway. In only a few special cases may the ambient conditions be controlled (or minimized) sufficiently to permit an evaluation of existing theories.



Theories

The approaches followed in attempting to derive a theory of traffic flow are legion (12). Early models were developed primarily to provide safe use of the highway system. These encompass the so-called "safe" distances - such as, a safe distance between successive vehicles* - that is, the driver of a "following car" trailed the "lead car" by a distance adequate for stopping without hitting the lead car (13). Once this distance had been established for the desired highway speed, capacity and density were calculated on this basis. Unfortunately, typical driver response to a lead vehicle did not always provide for a "safe" distance between vehicles. To provide more realistic models various other assumptions and refinements have been used with the "car-following" techniques. These include: (a) Obedience to a suggested fifteen foot spacing for each ten mph increment of velocity (14); (b) The following car accelerates proportionally to the difference in vehicle velocity at a previous reaction time interval (15); (c) The velocity of the following vehicle is determined by the space between vehicles, the velocity of the lead car and a reaction time lag (16); (d) The following car velocity is affected by acceleration noise (17); (e) Drivers maintain a minimum fixed distance between cars with vehicle speed being the minimum of "desired speed" and lead vehicle speed (18). The "car-following" theories have been verified only in special cases - such as tunnels - where it is possible to regulate vehicular speeds and spacings (19).

The second group of traffic flow theories result from analogy to other physical systems. Many of the same limitations - no passing, no overtaking - apply to this group as to the car-following theories. In fact, one author goes to the extent of listing many of these procedures as "car-following

* Other safe distances include the horizontal and vertical sight distance.

in the large" (12). In this grouping traffic flow is considered as being analogous to the flow of a one-dimensional fluid (20, 21) or a gas (22). Here the vehicles are not considered individually but comprise particles in a moving stream. The introduction of a "shockwave" or a "kinematic wave" is a dominant feature in these theories (20, 23, 24). Briefly the waves are induced by the progressive changes in acceleration which are propagated through a line of vehicles when the lead vehicles make a velocity change. Results of traffic studies in tunnels have revealed that the "kinematic wave" conditions generally do reflect conditions at bottlenecks but not in areas of less restricted flow (25).

A third traffic mechanism is the so called "steady-state" approach. This method relates the vehicle mass, the acceleration of the following car, a driver sensitivity factor, relative velocity of the two vehicles, and a driver-vehicle response time lag (26, 27). Although parameters have been evaluated, evidence of actual usage of a "steady-state" model is lacking.

The development of high speed large memory capacity computers have provided means of simulating traffic flow (2, 28). At present, this process is limited to only short section of roadways (29, 30) or at most to one isolated intersection (31). Vehicle arrivals are predicted by Poisson's distribution (32, 33, 34) or by a modified binomial distribution (35). The results of simulation studies have provided an equation for delay at signalized intersections (36) and warrants for types of traffic control devices at intersections (35).

Other advocated - although less exploited - approaches include a frictional force concept (37), the use of queueing theory (38), and the applicability of network theory to suitable traffic problems (39).

Statistical Studies

To evaluate the parameters of the equations derived by any of the previously mentioned theories, the experimenters have reverted to the use of statistical observations. That is, traffic flow is considered a random or stochastic phenomenon, and the methods of probability and statistics have been incorporated into the evaluation of the necessary parameters.

In 1934, Greenshields (40) published the results of a study made of a highway facility in Ohio. On the basis of his observations he concluded that the distribution of speeds were such that they plotted as a straight line on normal probability paper and that a straight line relationship existed between vehicle speed and vehicle density.

Other studies have indicated that distribution of instantaneous speeds of motor vehicles are approximately normal (11, 41, 42). The relationship between speed and density has proven satisfactory in special circumstances (19) although Edie and Foote (25) indicated that the straight line relationship is restricted to low concentrations of flow.

There have been in the past several statistical studies of the relationship between vehicle speed and some geometric feature of the highway. However these studies were primarily interested in demonstrating that the geometric aspect affected the speed distribution upon the highway and made no attempt to provide quantitative information of the resistance offered by the particular feature. The conclusions of these investigations show a statistical significant reduction in speed due to curvature (41, 43, 44), grades (41, 44, 45, 46, 47), and restricted sight distance (44, 48).



Quantitative Evaluation of Features

The quantitative evaluation of the speed of vehicles on a highway and of vehicular response to geometric aspects has only recently received any attention. The first such investigation was reported by Greenshields (49) in 1955. As a result of his studies, Greenshields was able to develop a so-called "quality index" rating. This index is a dimensionless number derived from the observed factors that cause inconveniences in driving, namely, number of speed changes, total amount of change in speed, and average speed. Greenshields' advocated usage of the "quality index" was in the area of the economics of vehicle operation - such as the development of relative cost factors to express the efficiency of a street, and a correlation between the index and gasoline consumption. However, the method provides no means of rating individual features for design purposes.

A "direct evaluation" procedure has been advocated by Bruening and Bone (50) who wrote:

Geometric highway design is an act which depends largely, if not entirely, on judgment, - - - As long as there is no accepted method of measuring them (geometric designs) there is no positive way of differentiating between the merits of the various designs.

Essentially their method requires that an "experienced traffic engineer" observe actual traffic patterns on the aspect, and then to compare these patterns with the desired patterns and make a rating based upon their differences. The resulting rating is strictly qualitative (provides no quantitative interpretation of the observations) and will likely be biased by the experience and training of the observer.

The most recent approach to traffic flow theory and the rating of geometric features was published by Harr and Leonards (10) 1962. They

postulated that the movement of traffic results from a motivating "pressure potential" - that is, the vehicle velocity between two points is a function of the difference in the potentials existing at these points. This is analogous to the fluid dynamics relationship between flow velocity and velocity potential, and similar to the relationship between gravitational force per unit mass and gravitational potential. When completely developed, this method results in a parameter (called the modulus of geometric aspects) which relates vehicle speed to the restriction generated by a geometric feature on the highway. The parameter is determined by measurement and is therefore unbiased and free from individual opinion.

The evaluation of this method is the primary objective of this study. Hence the derivation and use of the modulus of geometric aspects will be discussed more fully on the following pages.

THEORY

Assumptions

The development of the "modulus of geometric aspects" (see page 9) is based upon the following assumptions:

(a) The behavior of a vehicle upon a roadway is the result of the driver's reaction to a motivating "pressure potential" under the prevailing ambient conditions. The pressure potential p is defined so that:

$$\frac{\partial p}{\partial x} = -kv \quad \text{Eq. 1*}$$

(b) The change in vehicular density with respect to the pressure potential is proportional to the density:

$$\frac{\partial \rho}{\partial p} = c_1 \rho \quad \text{Eq. 2}$$

From Eq. 2

$$\frac{1}{\rho} \frac{\partial \rho}{\partial x} = c_1 \frac{\partial p}{\partial x} \quad \text{Eq. 2A}$$

and

$$\frac{1}{\rho} \frac{\partial \rho}{\partial t} = c_1 \frac{\partial p}{\partial t} \quad \text{Eq. 2B}$$

(c) The change in effective width of the roadway with respect to the pressure potential is proportional to the effective width:

$$\frac{\partial w}{\partial p} = c_2 w \quad \text{Eq. 3}$$

* See Appendix I for nomenclature.

From Eq. 3

$$\frac{1}{w} \frac{\partial w}{\partial x} = c_2 \frac{\partial p}{\partial x} \quad \text{Eq. 3A}$$

and

$$\frac{1}{w} \frac{\partial w}{\partial t} = c_2 \frac{\partial p}{\partial t} \quad \text{Eq. 3B}$$

Formulation of Problem (*)

Traffic flow is considered as a conserved system - that is, the change in the number of vehicles within a section, during a specified time interval dt , must equal the difference in the number of vehicles entering the section at position x and the number leaving the section at position $x + dx$ (during time interval dt). This is expressed symbolically by

$$\frac{\partial (\rho w)}{\partial t} dt dx = \rho w v dt - \left[\rho w v dt + \frac{\partial (\rho w v)}{\partial x} dx dt \right]$$

or

$$\frac{\partial (\rho w)}{\partial t} + \frac{\partial (\rho w v)}{\partial x} = 0 \quad \text{Eq. 4**}$$

By the use of equation 1, 2A, 2B, 3A and 3B and noting that $\left(\frac{\partial p}{\partial x}\right)^2$ is of higher order, equation 4 reduces to:

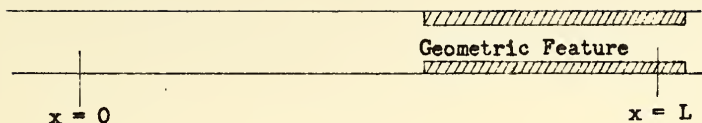
$$\frac{\partial^2 p}{\partial x^2} = a^2 \frac{\partial p}{\partial t} \quad \text{Eq. 5}$$

Application

For the solution of equation 5, a set of three boundary and initial conditions are required. For this purpose consider a section of roadway as shown in Fig. 1. At $x = 0$, the vehicle is unhampered by the geometric feature and is operating at a potential p_0 ($p(0, t) = p_0$). At $x = L$, —

* For more detailed derivation, see Reference 10.

** This equation is developed for the condition of no egress or access other than at the ends of the section under consideration.



Schematic Illustration of Roadway Section

Figure 1

a location influenced by the geometric feature — the potential has changed to p_1 $[p(L, t) = p_0 \pm \Delta p = p_1]$. At time $t = 0$, the drivers of the vehicles are unaware of the necessity of a change in potential on the roadway ahead thus $p(x, 0) = p_0$. With these conditions the complete solution of equation 5 is:

$$p(x, t) = p_0 + \frac{\Delta p}{L} x - \frac{2}{\pi} \Delta p \sum_{n=1}^{\infty} (-1)^{n-1} e^{-n^2 F_0} \sin \frac{n\pi x}{L} \quad \text{Eq. 6}$$

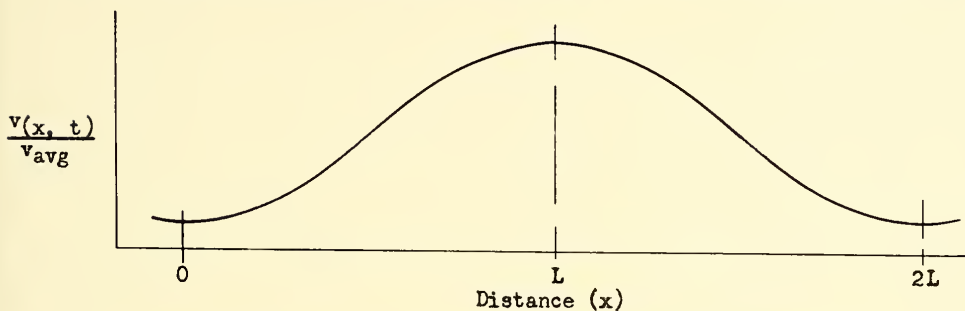
where $\Delta p = p_0 - p_1$ and $F_0 = \frac{\pi^2 t}{a^2 L^2}$

The introduction of equation 1 permits the pressure potential factor to be eliminated so that;

$$\frac{v(x, t)}{v_{avg}} = 1 - 2 \sum_{n=1}^{\infty} (-1)^{n-1} e^{-n^2 F_0} \cos \frac{n\pi x}{L} \quad \text{Eq. 7}$$

DEVELOPMENT OF THEORY

Figure 2 shows graphically the relationship between speed and location as computed by the mechanistic model derived in THEORY. The speed



Graphical Portrait of Theoretical Speed Ratio Distribution

Figure 2

relationship $(v(x, t) / v_{avg})$ is symmetrical about the location $x = L$ and repeats in intervals of $2L$. The ratio increases as x goes from 0 to L and decreases as x goes from L to $2L$. In the evaluation of specific geometric aspects vehicle speeds will normally decrease as the feature is approached because traffic flow is impeded. For these features the theoretical curve from $x = L$ to $x = 2L$ (area where the ratio is decreasing) would be considered. Rewriting the speed ratio equation so that the origin is moved L units to the right - replacing x by $(x + L)$ - gives

$$\frac{v(x, t)}{v_{avg}} = 1 - 2 \sum_{n=1}^{\infty} (-1)^{n-1} e^{-n^2 F_0} \cos \frac{n\pi}{L} (x + L) \quad \text{Eq. 8}$$

Where

$$\cos \frac{n\pi}{L} (x + L) = (-1)^n \cos \frac{n\pi x}{L}$$

Substituting this equality back into the speed ratio equation produces:

$$\frac{v(x, t)}{v_{avg}} = 1 + 2 \sum_{n=1}^{\infty} e^{-n^2 F_0} \cos \frac{n\pi x}{L} \quad \text{Eq. 9}$$

Figures 3 and 4 show the speed ratio-distance curves obtained by various values of F_0 (for numerical values see Table A1*). In Fig. 3, F_0 is limited to values above 0.6. Using only the first term of the series for the latter set produces the curves shown by dashed lines on the figure. When F_0 is above 1.0, the approximate curves and the complete series curves are indistinguishable. Table A2 lists the numerical error introduced with the approximate equation for selected values of F_0 . Using only the first term of the Fourier Series the speed ratio equation (Eq. 9) reduces to:

$$\frac{v(x, t)}{v_{avg}} = 1 + 2 e^{-F_0} \cos \frac{n\pi x}{L} \quad \text{Eq. 10}$$

which for $F = 1.0$ may be used without any significant error. The majority of the geometric features observed in this study allow the use of this simplified equation.

Rewriting the speed ratio equation so that the speed at any point is expressed in terms of the average speed results in:

$$v(x, t) = \bar{v} + 2\bar{v} \sum_{n=1}^{\infty} e^{-n^2 F_0} \cos \frac{n\pi x}{L} \quad \text{Eq. 11}$$

This plots into the general form shown in Fig. 5. On this figure v_0 is defined as the normal vehicle speed upon the highway under study, v_1 is

* Tables with an A prefixed to the table number are in Appendix II.

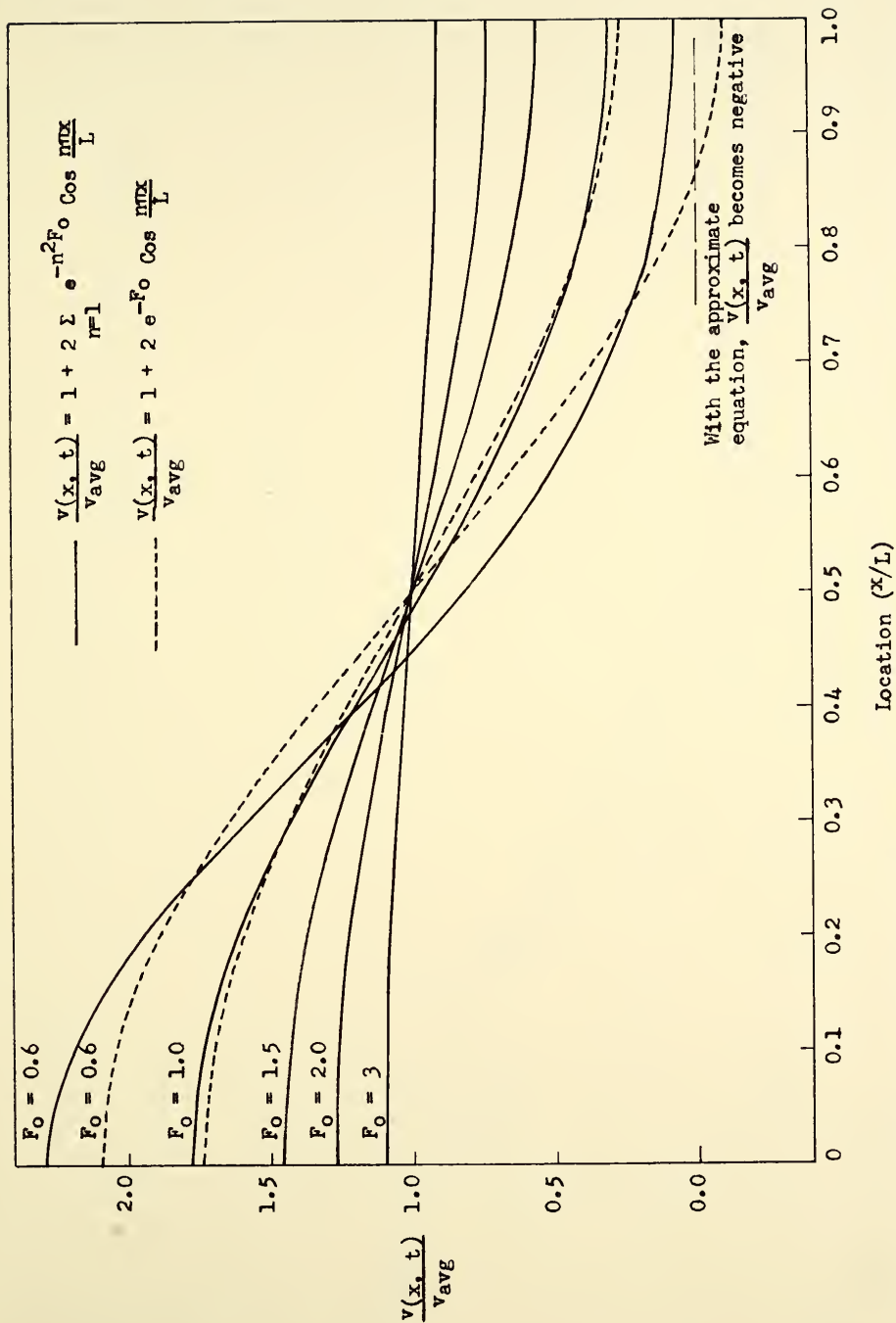


Figure 3. Theoretical Speed Ratio-Distance Relationships for Selected Values of F_0 0.6 and above.

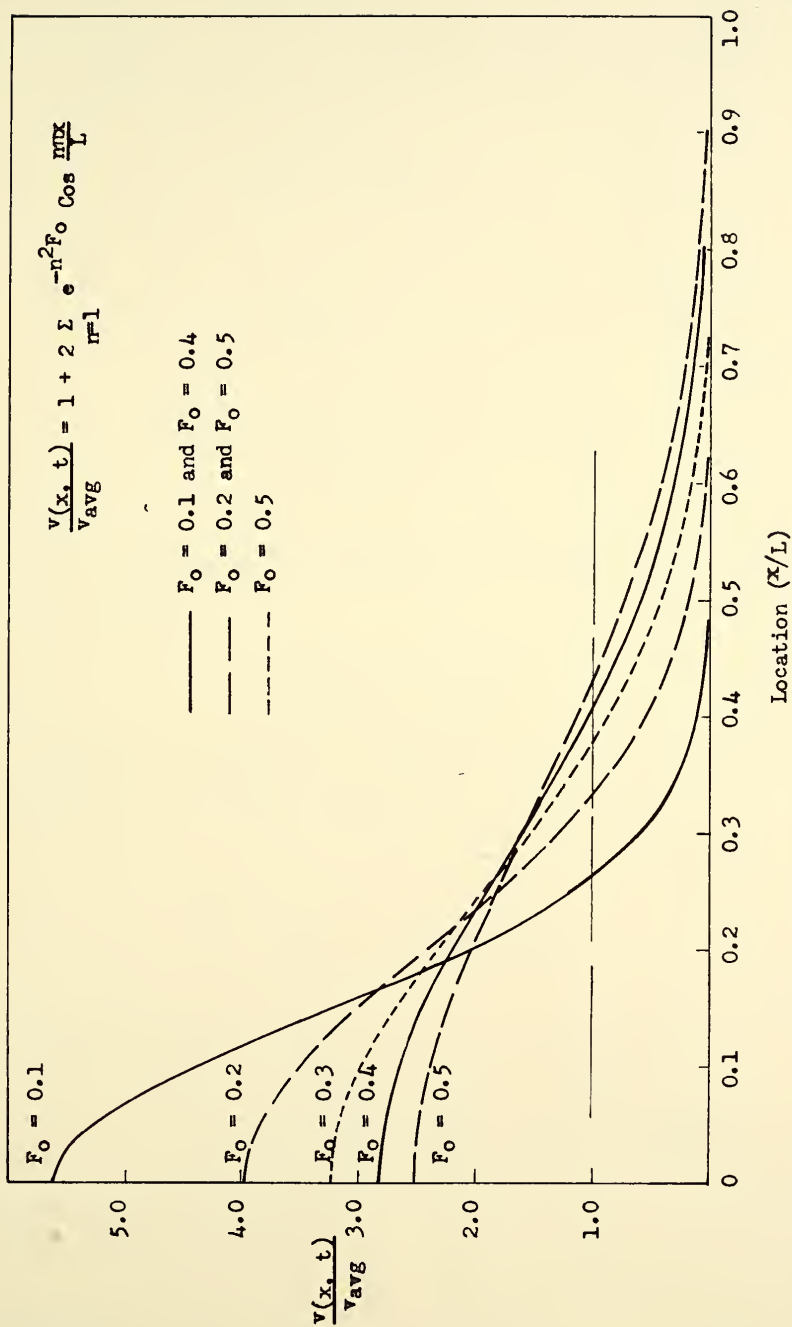
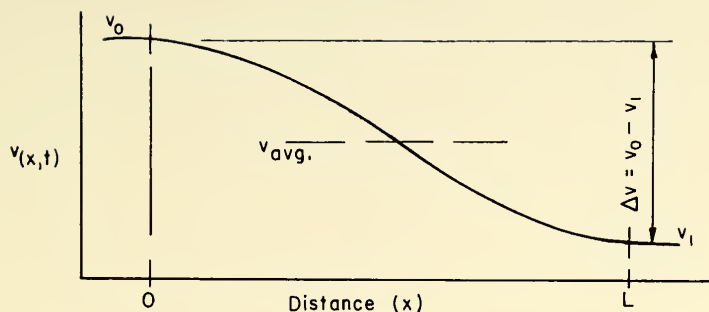


Figure 4. Theoretical Speed Ratio-Distance Relationships for Selected Values of F_0 below 0.6.



Theoretical Speed-Distance Relationship

Figure 5.

defined as the minimum speed caused by the influence of the particular geometric feature.

Approximate Solution

Using the approximation to equation 11 ($v(x, t) = \bar{v} + 2\bar{v} e^{-F_0} \cos \frac{\pi x}{L}$), the speed at any position x may be written:

$$v_x = \bar{v} (1 + 2 e^{-F_0} \cos \frac{\pi x}{L})$$

and
$$\bar{v} = v_x (1 + 2 e^{-F_0} \cos \frac{\pi x}{L})^{-1}$$

Selecting two positions, say $x = a$ and $x = b$, v_{avg} becomes:

$$\bar{v} = \frac{v_a}{1 + 2 e^{-F_0} \cos \frac{\pi x_a}{L}} = \frac{v_b}{1 + 2 e^{-F_0} \cos \frac{\pi x_b}{L}}$$

Solving these for e^{-F_0} we have

$$e^{-F_0} = \frac{v_a - v_b}{2 (v_b \cos \frac{\pi x_a}{L} - v_a \cos \frac{\pi x_b}{L})}$$

In particular, allowing x_a to equal 0 and x_b to equal L:

$$e^{-F_0} = \frac{v_0 - v_1}{2 (v_1 + v_0)} = \frac{\Delta v}{2 (v_0 + v_1)} \quad \text{Eq. 12}$$

Since $v_1 = v_0 - \Delta v$ this equation may be solved for F_0 as:

$$F_0 = \ln (4 v_0 - 2 \Delta v) - \ln \Delta v \quad \text{Eq. 13}$$

Equation 13 is plotted as Fig. 6 where F_0 may be read directly if Δv and v_0 are known and may be used for the calculation of F_0 providing F_0 is equal to or greater than 1.

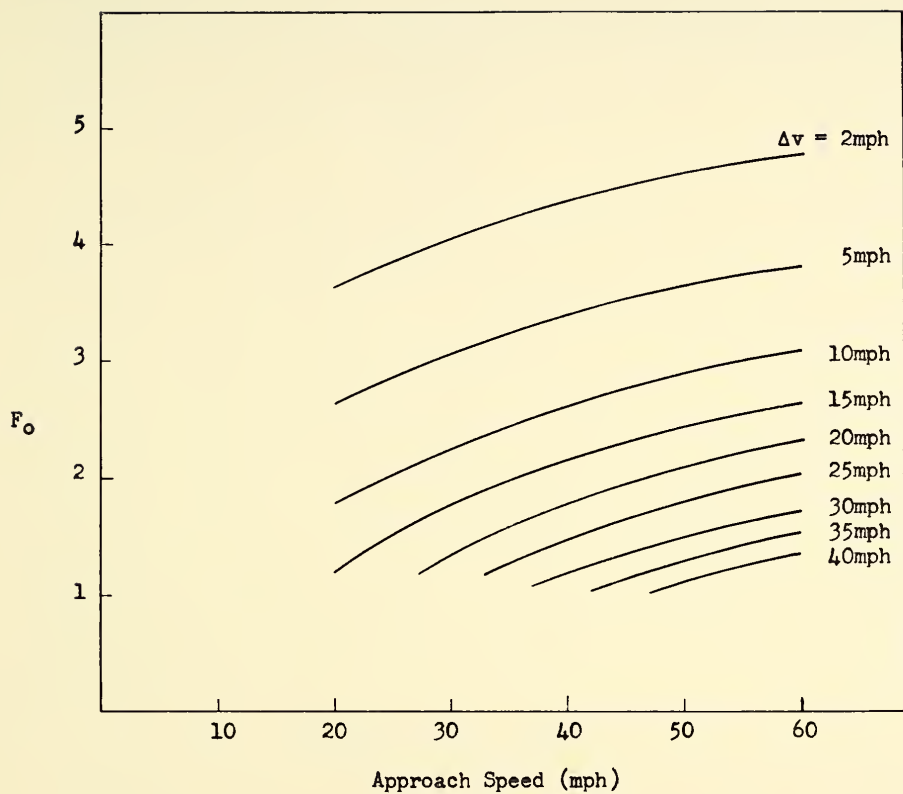


Figure 6. F_0 Graph for Selected Values of v_0 and Δv .

PILOT STUDY

General

During the review of literature, it became apparent that quantitative studies of the resistance offered to vehicular flow by geometric aspects of highways are nonexistent. For this reason, the form and amount of data required to determine the modulus of geometric aspects for use as a proposed rating system was unknown. To mitigate this difficulty it was decided that the study be undertaken in two parts. First, a pilot study of one geometric feature was made for the purpose of preliminary conclusions after detailed analysis of observed driver reactions. The second portion of the study extended these conclusions to other selected features where the results were observed and evaluated on a more general basis.

Procedure for Pilot Study

In the initial study of the proposed quantitative rating system a section of U. S. Highway 24 approximately one mile west of the town of Reynolds in White County, Indiana was selected (Fig. 7). The highway exhibits a $4\frac{1}{2}$ degree curve extending through a central angle of 45 degrees. The curve is level throughout, exhibits no degree of super elevation and has no apparent change in the road cross section. The west approach to the curve is a mile long straightaway with no perceivable grade. Visibility for the approaching drivers is unlimited. The section contained no

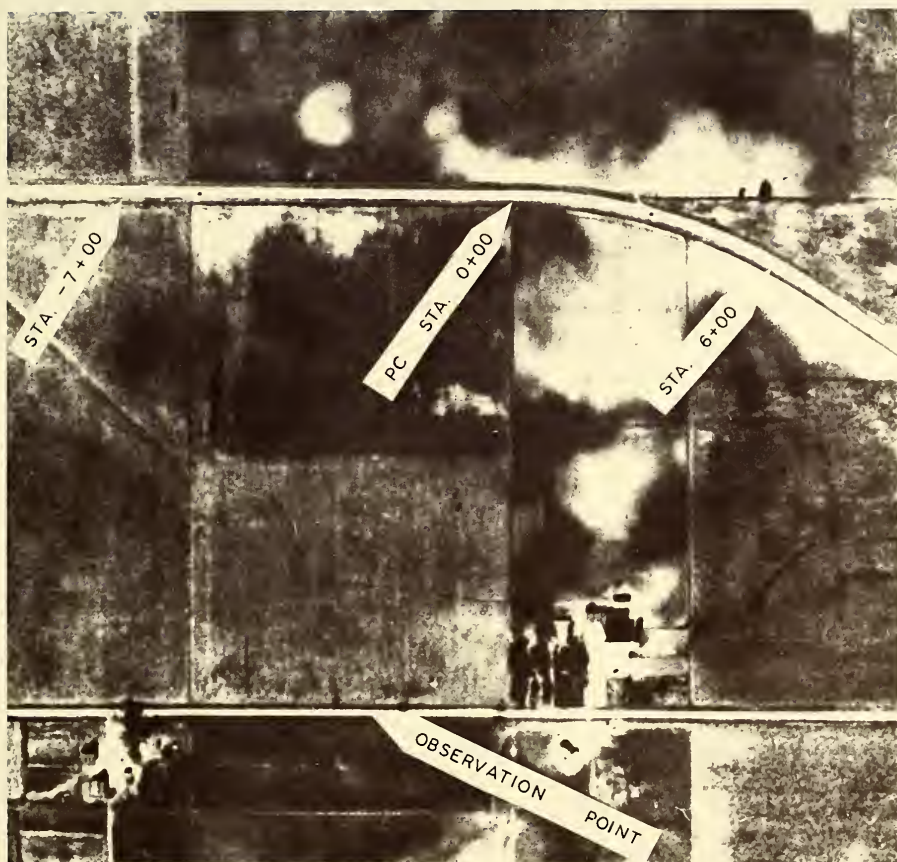


FIGURE 7. PILOT STUDY SITE

intersecting roads or driveways except for farm entrances to adjacent fields on the approach to the curve. During the periods of observation, no vehicle was seen entering, leaving or stopping in or near the study section.

The first two sets of data were collected on January 12 and January 20, 1962 respectfully. Initially a spring driven 16 mm motion picture camera equipped with a 4 inch telescopic lens was used to record the motion of the vehicles. All observations were made from a parallel county road located approximately one-quarter mile south of U. S. 24 from which the entire approach to the curve and the curve itself could be easily observed. As a check on the operating speed of the camera a stop watch was frequently photographed for several seconds. The first set of data (observations of 40 vehicles) was obtained when the roadway was partially covered with ice and packed snow. The second set of data (71 observations) was taken on a cold clear day when the highway surface was clear and dry, although the adjacent ground was snow covered.

For horizontal control, a separate series of photographs was taken with an assistant holding a marker at fifty foot intervals along the roadway. A strip mosaic was prepared from this group of photographs. By matching the salient features of the roadway on the mosaic with the projected images of the recorded data, it was possible to determine the location of the vehicle for any particular exposure. Successive photographs indicated the distance traveled by a vehicle during the interval of one frame. Using this increment of distance as a guide it was possible to estimate elapsed time to the nearest one-tenth of a frame.

The tabulation of the first two sets of data indicated some definite disadvantages to the procedure: (a) The spring driven camera was inconsistent in operating speed and required frequent recording of the stop

watch; (b) The use of the 50 foot film magazines resulted in extensive film waste due to frequent film changes; (c) The tedious matching of salient features on the mosaic with their projected images was time consuming and expensive.

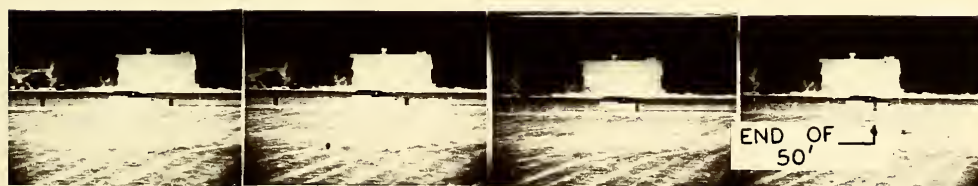
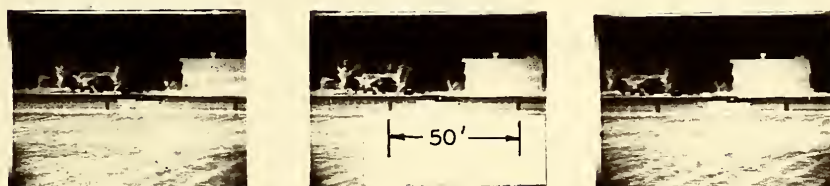
To mitigate these disadvantages, revisions in the method of observation were made. A battery powered 16 mm movie camera was borrowed from the Audio-Visual Department of Purdue University. This camera would operate with 400 foot film magazines which reduced film waste and was less expensive for equivalent amounts of film. Furthermore, the Audio-Visual Department had a large range of telescopic lens which allowed more flexibility in usage.

To expedite the tabulation of data, after the difficulties encountered by the earlier procedure, a system of markers were established along the right of way fence line. The placement of the markers was carefully chosen so as not to be noticed by drivers on the roadway. Fifty foot intervals were located along the travel lane of the roadway under observation. These distances were projected radially from the roadway to the fence line and were marked by 6 inch wide stakes driven in the ground. The stakes were painted white and set to reflect the sunlight so they could be easily observed on the projected image. From the point of observation, the passage of a vehicle from one stake to the next resulted in a traveled distance of fifty foot on the roadway. This enabled the observer to record the number of frames required for a vehicle to travel each fifty foot interval without using the mosaic (Fig. 8).

Four additional sets of data were recorded for the curve on U. S. 24 using the improved observation procedure. The third and fourth sets were



VEHICLE ENTERING 50 FT. SECTION



VEHICLE LEAVING 50 FT. SECTION

FIGURE 8. DATA AS RECORDED BY
THE PHOTOGRAPHIC METHOD

taken on March 28, 1962, a warm clear day. Data set 3 (50 observations) was collected at approximately mid-day while set 4 (50 observations) was obtained during late afternoon. The fifth set of data was taken during the early morning hours of April 14, 1962 and consisted of 31 observations. The last set of data (50 observations) was recorded on the afternoon of Sunday June 10, 1962. All of these data were photographed at approximately 25 frames per second which permitted an estimated time interval to within one-two hundred and fiftieth of a second.

The developed film was viewed by a time-motion study projector belonging to the Joint Highway Research Project, Purdue University. As a vehicle arrived at the first marker of the roadway section under observation the projector frame counter was set to zero. Then, as the vehicle crossed each succeeding 50 foot mark, the accumulated number of elapsed frames was recorded (estimated to nearest tenth of a frame). The difference between consecutive readings provided a time measure, in terms of the number of photographic frames, required for the vehicle to travel each fifty foot distance. Hence the number of frames traversed, divided by the camera operating speed (frames per second) resulted in elapsed time in seconds. Thus a known distance (50 foot in this study) and a corresponding travel time in seconds was obtained for each vehicle from which "spot" speeds were calculated on the approach to and passage through the geometric aspect under consideration.

Data Preparation

After the data had been tabulated they were separated into observations of passenger cars and observations of trucks because the two have different

operating characteristics*. Next the recorded data were reduced to vehicle speeds by the LGP-30 computer of the Civil Engineering School at Purdue University. The initial calculation divided the number of elapsed frames per distance interval by the camera operating speed to produce a time interval in seconds. This was converted into an interval of distance to give speed which was then converted into units of miles per hour. The speeds were summed and averaged for each position on the roadway (Table A3). These speeds were then plotted versus location and Figs. 9 through 15 inclusive resulted.

Visual inspection of the shape of the measured speed location curves indicated a striking similarity to the theoretical curve of Fig. 5. Quantitative curve fitting of the observations to the theoretical curve - and hence an estimate of the modulus of geometric aspects, F_0 - was obtained by an application of the principles of least squares. These calculations, as programmed for the LPG-30 computer, consisted of (a) assigning various locations for the start and end of the theoretical curve; (b) obtaining v_0 and v_1 as the average speed prior to the start and subsequent to the end of this curve respectively and using these values as constant speeds beyond the ends of the theoretical curve; (c) evaluating the theoretical speed for each increment of distance to correspond with the field observations; (d) subtracting the theoretical speed from the observed speed, squaring this difference, and summing the squared values; and (e) obtaining as output the start of curve x_0 , end of curve x_1 , speed prior to the curve v_0 , minimum speed in the geometric feature v_1 , the damping factor e^{-F_0} , and

* Because of the extreme differences in the speed characteristics between passenger cars and trucks, in no case were the data for these vehicles combined; that is, in the subsequent development a complete separation is made between the two.

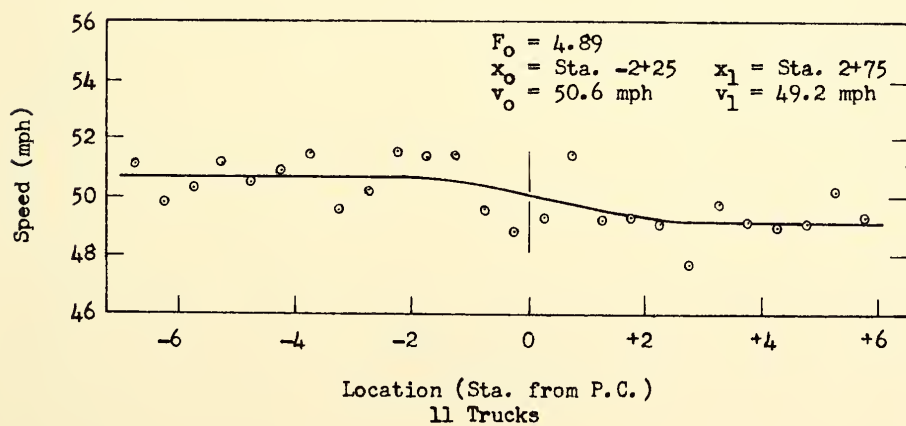
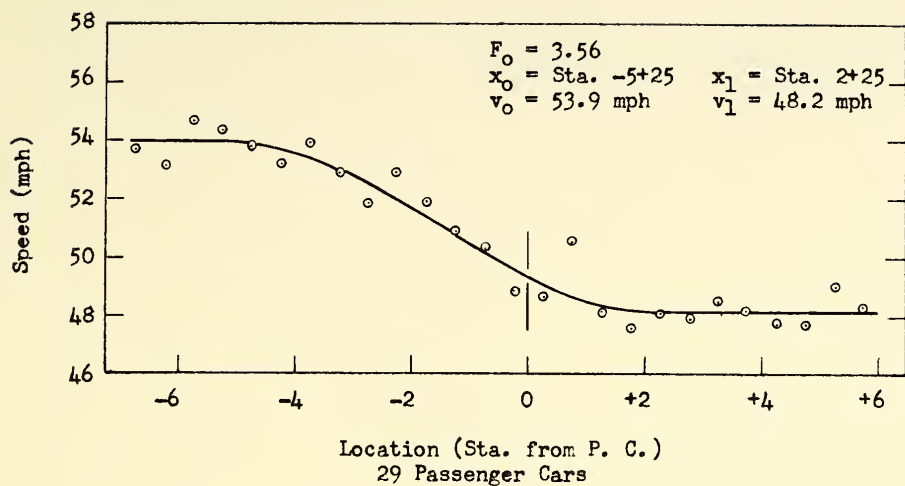


Figure 9. Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana

Data Set Number 1

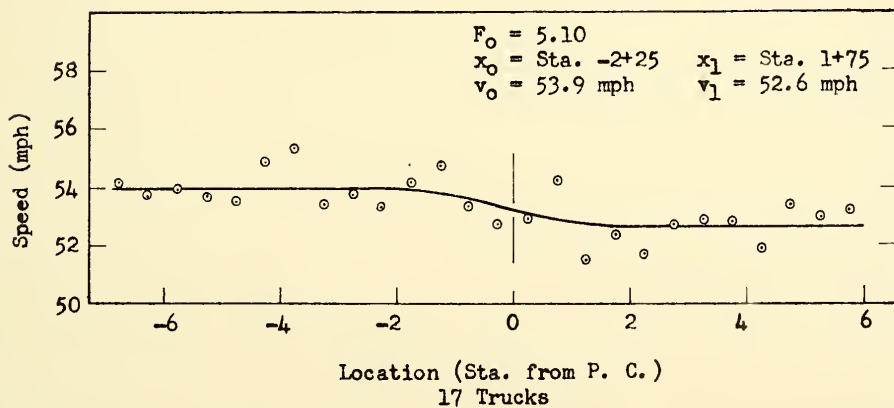
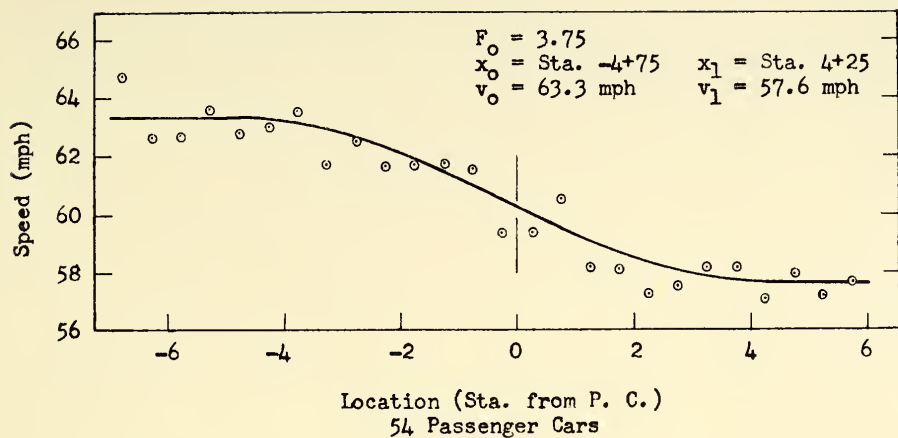


Figure 10. Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana
Data Set Number 2

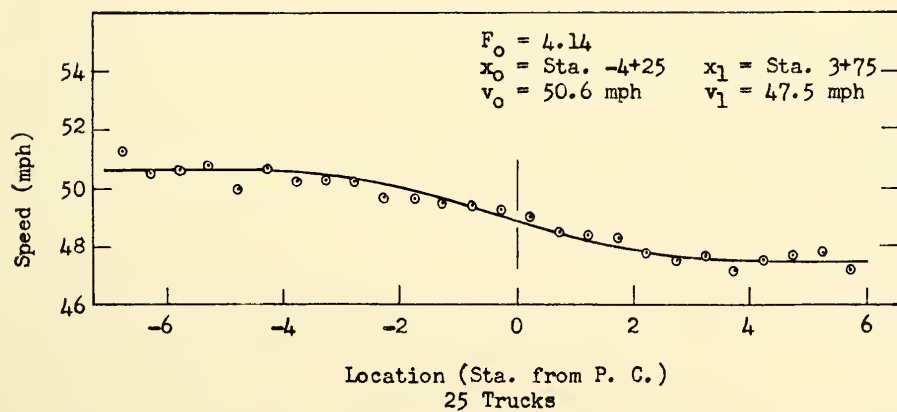
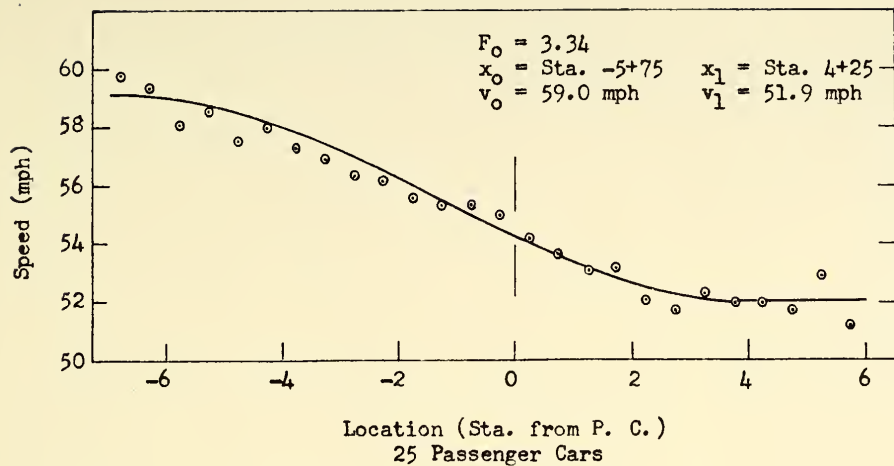


Figure 11. Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana

Data Set Number 3

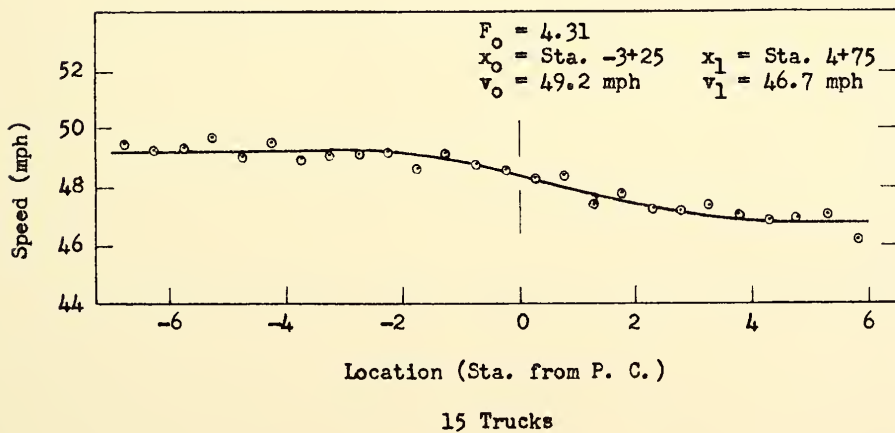
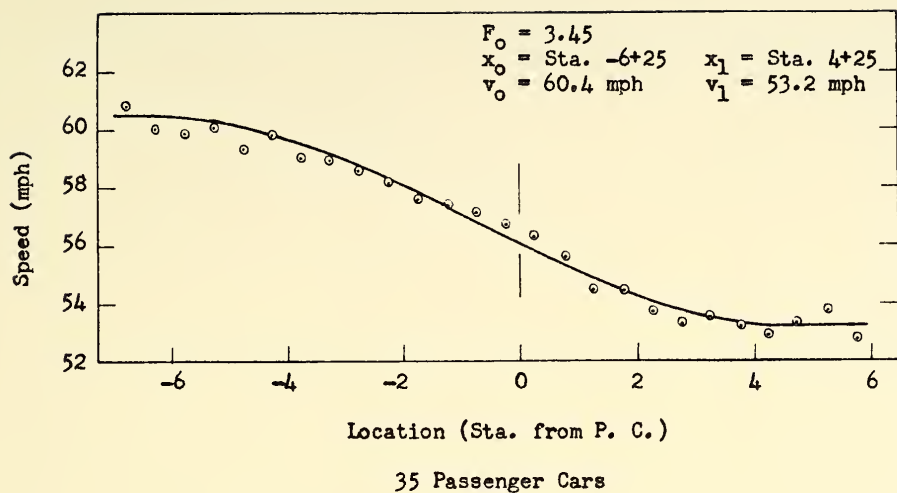


Figure 12. Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana

Data Set Number 4

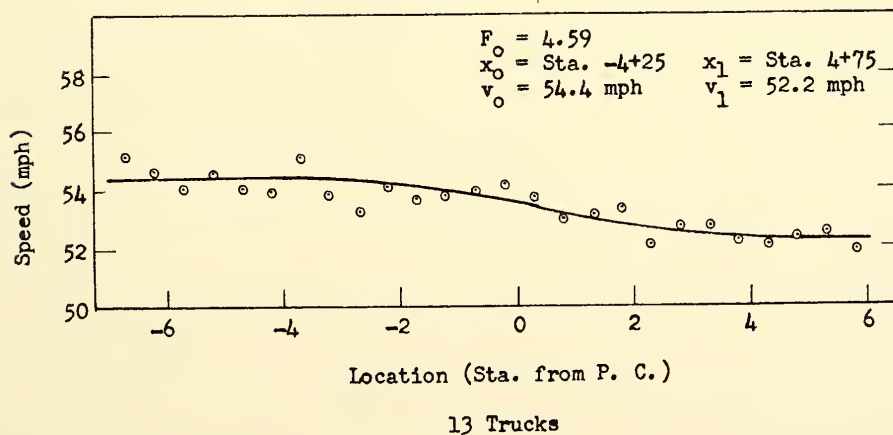
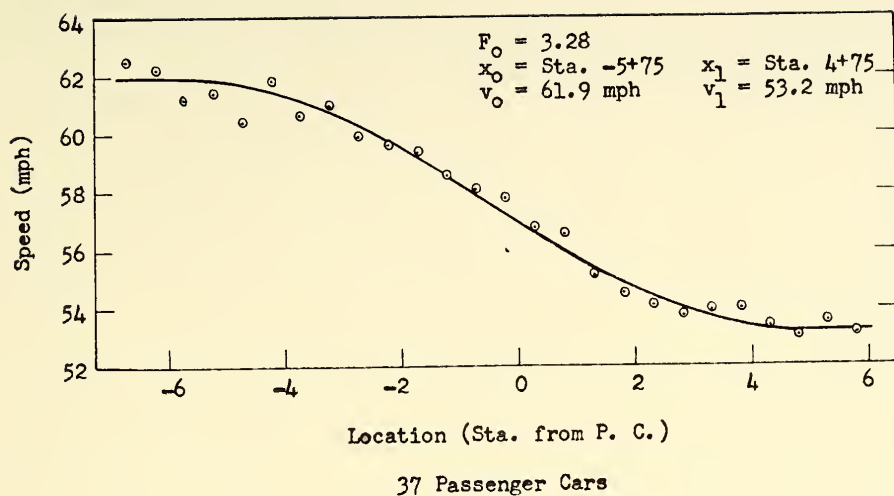


Figure 13. Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana

Data Set Number 5

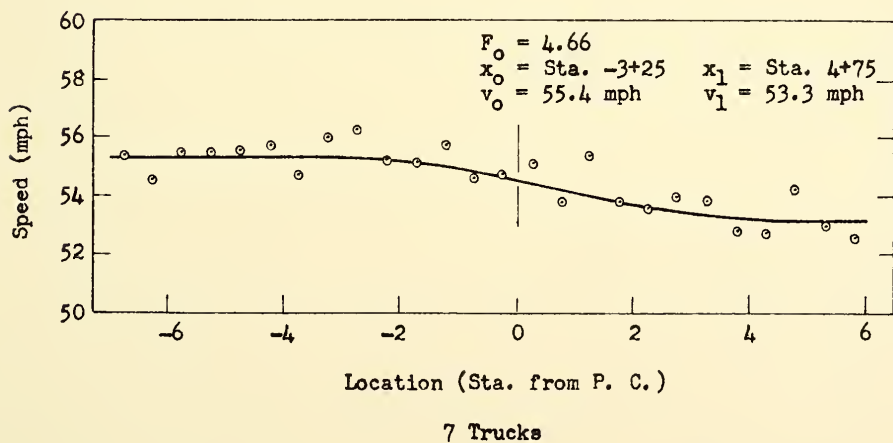
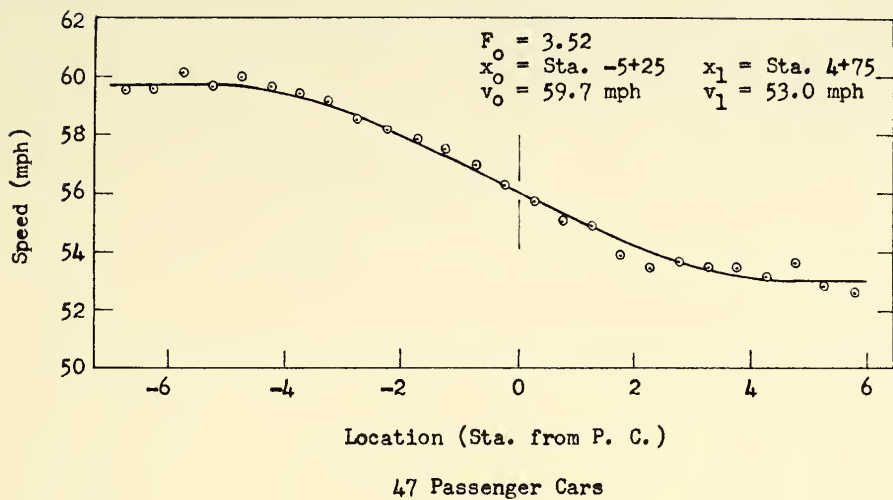


Figure 14. Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana

Data Set Number 6

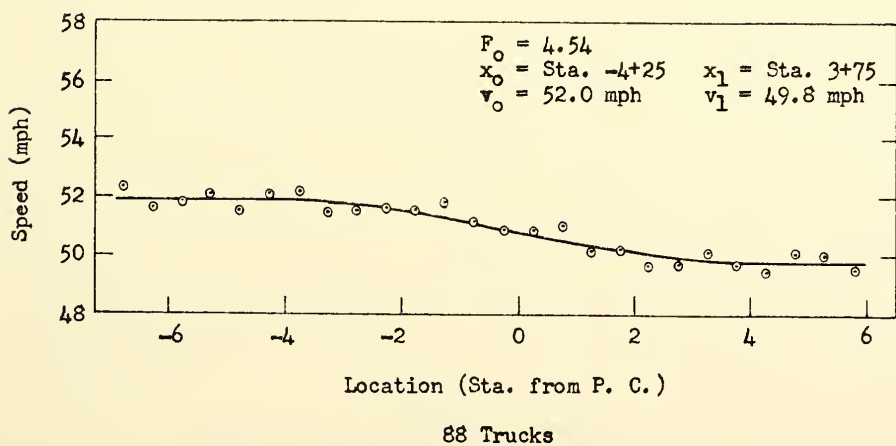
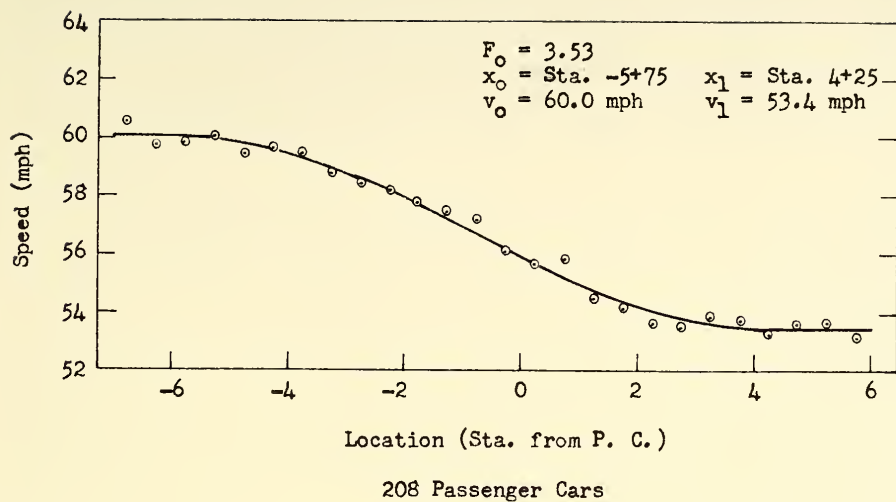


Figure 15. Speed-Location Graph for Curve on U. S. 24 near Reynolds, Indiana

All Data Sets

the sum of squared difference $\Sigma(v_0 - v_1)^2$. The curve which "best fit" the observations and which was of the type of the theoretical curve was determined by selecting the minimum value of the sum of squared differences. The "best fit" curve for each set of data is drawn on Fig. 9 through 15 inclusive.

After selection of the "best fit" theoretical curve, the locations x_0 and x_1 were used to calculate for each individual vehicle the values of v_0 , v_1 , $v_0 - v_1$, $(v_0 - v_1)(v_0)^{-1}$, e^{-F_0} , and F_0 . These computations were programmed for the LGP-30 computer and were accomplished in much the same manner as described above. A frequency distribution of each of these factors is provided in Tables A4 through A9 inclusive.

Data Analysis Procedure

Fit of Theoretical Curve

The ability of the proposed pressure potential theory to explain the observed speed changes on the approach and through the geometric feature was evaluated by whether or not a significant amount of variability in speeds was accounted for by the "best fit" theoretical curve. This is the statistical method of testing the significance of a regression model. The test is illustrated by Table 1. In this table, S.S. $_{\tau}$ (total sum of squares)

TABLE 1
ANALYSIS OF VARIANCE FOR THE SIGNIFICANCE
OF THE PRESSURE POTENTIAL THEORY

Description or Source of Variance	Degrees of Freedom	Sum of Square	Mean Square
Removed by Theory	2	S.S. $_{\text{R}}$	$\frac{\text{S.S.}_{\text{R}}}{2} = \text{M.S.}_{\text{R}}$
Unexplained	n-3	S.S. $_{\text{U}}$	$\frac{\text{S.S.}_{\text{U}}}{n-3} = \text{M.S.}_{\text{U}}$
Total	n-1	S.S. $_{\tau}$	

is the sum of the squares of the difference between the average observed speed at each location and the overall average speed; S.S._U (unexplained sum of squares) is the sum of squared differences between the observed and theoretical speeds remaining after the curve has been fitted - previously calculated as $\sum (v_0 - v_1)^2$; and S.S._R (sum of squares removed) is the amount of variability in the observed speeds removed by the theoretical curve and is equal to S.S._T - S.S._U. The value of n is the number of locations in the roadway section under consideration. There are two estimated values in the theoretical curve (the parameter F_0 and the curve length L) and thence two degrees of freedom were used for the description or source of variance called Removed by Theory in Table 1. The actual test of significance is based upon the ratio of M.S._R (mean square removed) to M.S._U (mean square unexplained) and is distributed as the ratio of two Chi squares (or the statistical F distribution).

Individual Vehicle Statistics*

The normality of the distribution of the statistics calculated from the characteristics of individual vehicles was tested by plotting accumulated percentages of these factors on probability paper. In some special instances the Chi-squared test of goodness of fit was used. In addition to these, the Bartlett test was used to check homogeneity of variance between data sets. The difference between means of data sets was checked by an analysis of variance for a single variable of classification. Each set of data for the highway curve on U. S. 24 was considered as being representative of traffic response to the feature under the different ambient conditions.

* The procedures used in the statistical testing program were consistent with the generally accepted methods described by Dixon and Massey (51).

Results

Fit of Theoretical Curve

The speed variability removed by the pressure potential theory is extremely high*. The lowest statistical F value (Table 2) for any of the data sets for the U. S. 24 highway section is 8.9 (trucks of data set number 1) which is significant at the 0.005 level of probability - that is, only five times in a thousand would this amount of variability be removed by pure chance. The other data sets - for both passenger cars and trucks - indicate an even better fit of the theoretical curve to the observed data. Therefore it must be concluded that all the data exhibits definite agreement with the advocated theory.

TABLE 2
STATISTICAL F VALUES FOR THE SIGNIFICANCE
OF THE PRESSURE POTENTIAL THEORY

	Passenger Cars		Trucks	
	Statistical F	(R ²)	Statistical F	(R ²)
Data Set 1	68.5	(85.6)	8.9	(43.6)
Data Set 2	137.0	(92.2)	10.7	(48.2)
Data Set 3	245.0	(96.4)	214.4	(95.0)
Data Set 4	573.1	(98.0)	157.5	(93.0)
Data Set 5	622.0	(98.2)	41.8	(78.6)
Data Set 6	1016.0	(98.9)	28.6	(71.4)
All Data	949.1	(98.8)	131.6	(91.9)

* R² as listed in Table 2 provides an estimate of the percentage of the variability removed by the pressure potential theory.

Open Highway Speed (v_0)

Vehicle speeds of both passenger cars and trucks prior to the influence of the feature (Table A4) followed the generally anticipated pattern. They may be considered as normally distributed, their variances appear to be homogeneous for all sets of data while the average speed showed a significant difference among the six data sets. This indicates the stochastic distribution mentioned in the Review of Literature.

Speeds within the Feature (v_1)

Individual vehicle speeds within the feature (Table A5) followed the same general pattern as open highway speeds. They also appear to be normally distributed, the variances for each type of vehicle were homogeneous, and average speeds were significantly different for the diverse data sets. The variance of v_1 was significantly smaller than the variance of v_0 , thus indicating a definite narrowing of the range of speeds within the influence of the highway curve.

Speed Changes ($v_0 - v_1$)

The speed changes occurring on the approach to and within the feature (Table A6) produced a definite pattern. First the variance within data sets by type of vehicle was statistically the same for all data sets. Second the mean speed change for a vehicle type was statistically the same under all observed ambient conditions. Third these changes followed a normal distribution. The interpretation of these observations indicates that a particular type of vehicle may tend to reduce speed the same amount at a specific geometric feature irrespective of roadway and other conditions.

$$\text{The Ratio } \frac{v_0 - v_1}{v_0}$$

One of the original factors which was considered as a possible representative value of driver response to geometric aspects was the ratio of speed change to open highway speed (Table A7). The statistical analysis of these values indicated no common value between the various data sets. Consequently the ratio was considered of no relevance in this study.

$$\text{The Term } e^{-F_0}$$

Because the fundamental purpose of this study was to verify the use of the modulus of geometric aspects (or F_0), it was natural to investigate the distribution of the term e^{-F_0} (Table A8) - the so-called speed dampening factor. The variance of the factor is non-homogeneous between data sets for passenger cars and homogeneous between data sets for trucks. Conversely the mean value of e^{-F_0} appears to vary between data sets for trucks although passenger car data indicate no change between data sets (using the pooled variance regardless of the non-homogeneity). Therefore the speed dampening factor shows little possibility of being an efficient indicator of driver response to the geometric feature.

Modulus of Geometric Aspects (F_0)

As evidenced by the distribution of speed changes (Δv or $v_0 - v_1$) a few vehicles increased rather than decreased speed on the roadway section studied. In order to symbolize this reversal when considering F_0 for individual vehicles, it became mathematically expedient to assign to F_0 a negative value for such cases (Table A9). Because F_0 as obtained from individual vehicles is given as a function of the natural logarithm of the speed change (Eq. 13) and the speed change distribution was shown to

be normal the distribution of F_0 for individual vehicles can not be normal. The study of speed changes reflects a study of the individual F_0 values under a required transformation to obtain a normal distribution. Therefore the individual F_0 distribution was not analyzed beyond the tabulation of the values.

Although F_0 values as calculated from observations of individual vehicles do not represent the flow restriction, the F_0 as calculated from the average of several vehicle responses does provide a measure of flow restriction (as evidenced by the fit of the observed data to the theoretical curve). Based upon the fit of the theoretical curve to the average of all observations, the highway curve on U. S. 24 has an F_0 of 3.53 for passenger cars and 4.54 for trucks.

To secure an estimate of the number of observations required to estimate F_0 , it is necessary to consider Eq. 13. With the conditions indicated for the geometric feature on U. S. 24 - that is, average v_0 of 60 mph, average speed change of 6.36 mph, and a standard deviation (for the speed change) of 4.36 mph - 25 passenger cars will provide an estimate of F_0 within plus or minus 0.6 at a 95 percent confidence level*. For trucks - average v_0 of 52 mph, average speed change of 2.26 mph with a standard deviation of 3.04 - 16 observations would produce an estimate within plus or minus 0.65 at a 90 percent confidence level*. It is recommended that a minimum of 25 passenger cars and 16 trucks be used to evaluate F_0 from the average observed values.

* These limits are not strictly symmetrical because the logarithm varies more with a unit change in Δv when Δv is small than when Δv is large.

Summary and Recommendation

The modulus of geometric aspects provided a quantitative rating for the geometric feature of the Pilot Study when calculated from the average speeds of at least 25 passenger cars or at least 16 trucks. When working with individual vehicles, the speed change attributed to the geometric feature was a better representation of driver response to flow restriction.

On the basis of these findings, the study was extended to other geometric features in order to determine whether or not the proposed model - and its accompanying modulus of geometric aspects - may be applied in general to the geometric features of highways.

EXTENSION OF STUDY

Sites

On the basis of the findings of the Pilot Study, additional geometric aspects were selected for the extension of the study. The following features were chosen:

- (a) A $5\frac{1}{4}$ degree horizontal curve on U. S. 41 (and 52) located approximately one mile north of Earl Park, Indiana (Fig. 16, p. 44).
- (b) A right angle turn on Indiana 26 located approximately one mile east of Pine Village, Indiana (Fig. 18, p. 47).
- (c) A transition section from 4-lanes to 2-lanes on U. S. 52 north of Templeton, Indiana (Fig. 20, p. 51).
- (d) A merging lane on the North River Road entrance to the William Henry Harrison Bridge in West Lafayette, Indiana (Fig. 22, p. 53).
- (e) A narrow bridge on Indiana 43, located approximately two miles north of Chalmers, Indiana (Fig. 24, p. 56).

Field Procedure

The general procedure as developed in the Pilot Study was used to prepare the chosen sites for the recording of data. The common path of the vehicles, as determined from pavement wear and direct observation, was divided into 50 foot segments (except for the transition on U. S. 52 where 100 foot segments were used). These distances were projected radially to

the property line and marked with 6 inch wide stakes. As in the previous Pilot Study, this allowed the recording of the vehicles as they passed points of known location. Figures 16, 18, 20, 22, and 24 show the locations of the observation point and certain locations on the pavement for each site.

The forward motion of the vehicles was recorded by the 16mm movie camera which was again borrowed from the Audio-Visual Department of Purdue University (see page 22 for description). For these tests the operating speed of the camera was selected as 30 frames per second. This was frequently checked by observation of a stop watch.

Data was recorded on days of comparatively good weather - that is, for all observations the road surface was clear and dry. Only one period of observation was used for each of the selected sites as it was believed that broader coverage should be obtained at the sacrifice of information for fewer sites under varying ambient conditions. In all cases, each vehicle, for which data were recorded, could choose its own speed - there was no vehicle immediately (10 seconds or less) preceeding it in its particular traffic lane. However where possible the data were analyzed on the basis of encountering (or not encountering) vehicles traveling in the opposite lane. If an oncoming vehicle was met within the critical part of the feature, the data were classified as the opposing lane occupied. Otherwise the opposing lane was considered free of oncoming traffic (for example see Table A10).

Preparation of Data

The photographic record of data was transcribed visually into usable form and then all calculations were carried out on the LGP-30 computer.

The first calculation produced the spot speeds at the selected locations along the roadway. The second calculation permitted the selection of the "best fit" theoretical curve and also provided a measure of this fit. The third calculation resulted in the factors for individual vehicles. At each step, the required figures and tables were prepared for analyzing the data.

Curves

Curve on U. S. 41 (and 52)

Description. This curve (Fig. 16) is on a heavily traveled section of U. S. 41 and is the location of frequent accidents - some of which are severe. Both of the approaches are down grade to the curve and visibility is not restricted. The pavement has been widened for the inside (north-bound) lane. There is a minimum amount of super elevation through the facility. The approaches to the feature are marked by flashing amber caution lights and large signs proclaiming a dangerous curve ahead. Within the curve there is a minor county road intersecting the highway on the outside edge. During the period of observation no vehicle was seen using this minor facility. Data (Table A10) were recorded for vehicles traveling south (on the outside lane). Tabulation of calculated factors are listed in Table A11 through A16.

Results. The difference between the two best-fit theoretical curves shown on Fig. 17 is not significant. Therefore there is no evidence of a difference in driver response resulting from opposing traffic and all data may be considered as part of the same population - that is, the pressure of oncoming cars produces no significant change in driver response to the highway feature.



FIGURE 16. SITE OF CURVE
ON U.S. 41 (AND 52) NEAR EARL PARK, INDIANA

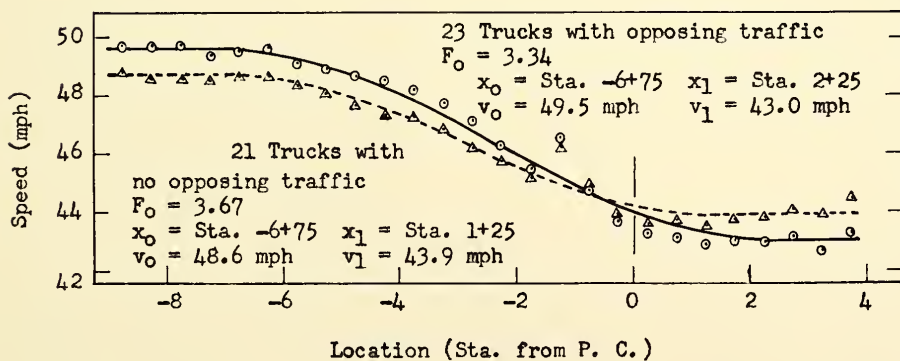
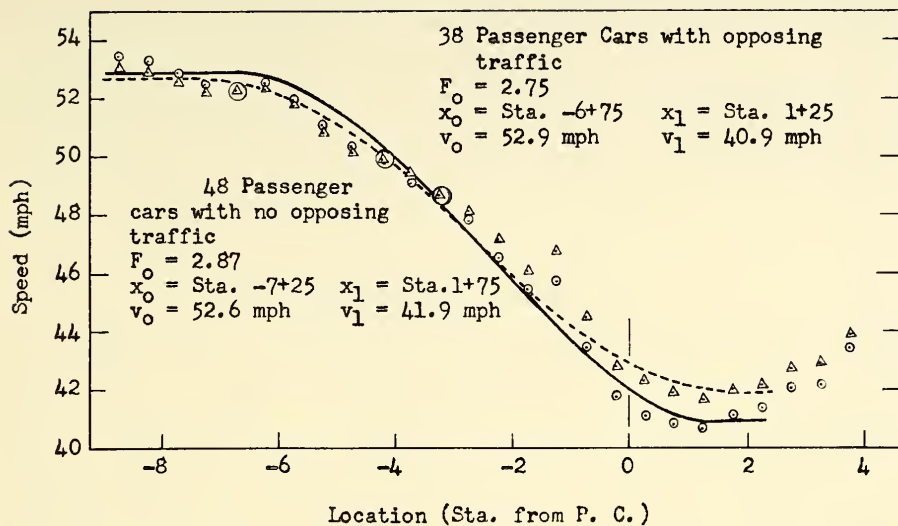


Figure 17. Speed-Location Graph for Curve on U. S. 41 (and 52) near Earl Park, Indiana

The speed variability removed by the advocated theory is again extremely high indicating an excellent fit between observation and theory. The distribution of speed changes plots as a normal distribution.

The best estimate of the modulus of geometric aspects for this highway curve from all observations is 2.78 for passenger cars and 3.42 for trucks.

Right Angle Corner on Indiana 26

Description. This feature is on a lightly traveled section of Indiana 26 (Fig. 18). However it is the location of frequent accidents, most of which are limited to minor damage. The approach to the turn is slightly down grade and visibility is not restricted. There is considerable banking of the asphalt surface in the turn. Approaching drivers are informed of the conditions by means of a sharp turn sign only.

The roadway makes a complete 90 degree bend with an estimated radius of 42 feet for the westbound travel lane for which observations were recorded. Thus this condition is an example of an extreme rural highway curve.

The speed data for this feature is listed in Table A17 and the calculated factors are listed in Table A18.

Results. The speed-location curve (Fig. 19) is not similar to the ones previously encountered in this study. Approaching vehicles begin to reduce speed approximately 1000 to 1200 feet prior to the turn as in the other curves studied. However in this case, vehicles continue their deceleration at a more progressive rate as the feature is approached. When the minimum speed is reached - at approximately the center of the turn - the vehicles immediately undertake an acceleration. This type of response to the feature differs from the pressure potential theory previously discussed.



FIGURE 18. SITE OF CORNER
ON INDIANA 26 NEAR PINE VILLAGE, INDIANA

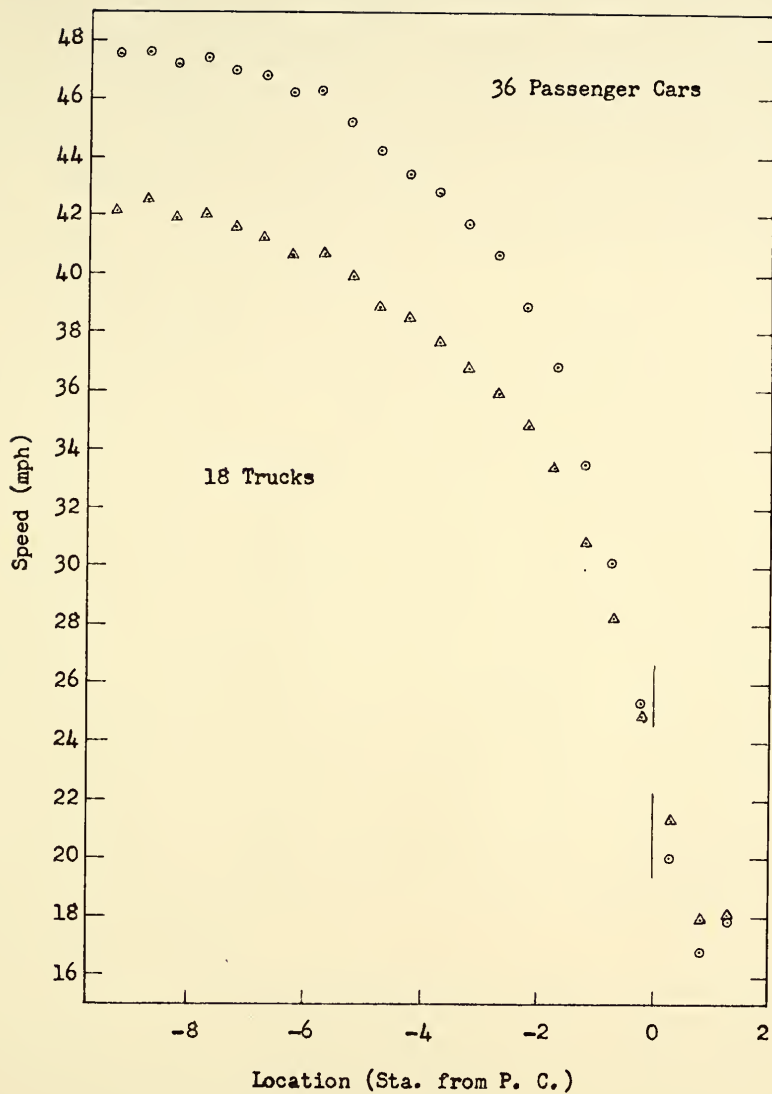


Figure 19. Speed-Location Graphs for Corner on Indiana 26 near Pine Village, Indiana



Matson et al (52) shows a comparable speed-location relationship which has been computed for the approaches to stop signs based upon a study by Beaky (53). The plotted observations for the subject curve on Indiana 26 resembles this condition more closely than the previous features in this study. Consequently the curve on Indiana 26 would be classified as a severe condition - that is it requires an extremely large change of speed to permit vehicles to safely traverse it.

The shape of the speed-location curve may be attributed to this severity of operation. As the driver approaches the turn, he becomes aware of the increased resistance and subsequently allows his vehicle to decrease speed. As the turn becomes closer, the potential changes and the driver must apply the vehicle's brakes to reduce speed more rapidly. Thus there are required at least two moduli of geometric aspects to be able to describe the response. An indication of this may be viewed in the similarity of the first 500 feet of deceleration shown by the observed data for all features. While the initial portion of the observed relationship is comparatively uniform, the remaining section exhibits extreme differences.

The detailed study of how to handle this particular response is beyond the scope of this study and will be left for future investigations into the pressure potential theory.

Transition Sections

U. S. 52 4-Lane to 2-Lane

Description. The feature under consideration is a moderately heavily traveled section of U. S. 52. For the vehicles observed (traveling north) the feature consists of a straight, level approach on the divided 4-lane

pavement, a curve to the left, a short straight section, and a curve to the right which brings traffic onto the 2-lane roadway (Fig. 20). Yellow paint has been applied to the pavement on the approach in order to encourage traffic to merge into a single lane prior to the transition. Warning and directional signs (for the transition) are located approximately 1000 feet prior to the actual feature. There is only a minor degree of super elevation evident on the curved portion.

Average speeds for the approach to and through the features are in Table A19. Frequencies of the calculated factors are listed in Tables A20 to A25 inclusive.

Results. Although the speed-distance charts (Fig. 21) indicated some difference between the speeds of vehicles encountering oncoming traffic and those not encountering such traffic, this difference was found to be statistically insignificant.

The speed variability removed by the theoretical curve is extremely high and an excellent fit is evident both visually and by statistical F tests. The speed changes for passenger cars were normal; however the speed changes for trucks was found to be skewed towards zero.

The best estimate of F_0 for this feature at the time of test was 2.76 for passenger cars and 3.69 for trucks.

Approach to Wm. H. Harrison Bridge

Description. In this study an investigation of the suitability of the modulus of geometric aspects was undertaken for the merging section of the North River Road approach to the Wm. Harrison Bridge in West Lafayette, Indiana. Observations were recorded of vehicles on the straight ramp approach, through the merging area, and onto the bridge itself (Fig. 22).





FIGURE 20. SITE OF 4-LANE TO 2-LANE
TRANSITION SECTION ON U.S. 52 NEAR TEMPLETON,
INDIANA

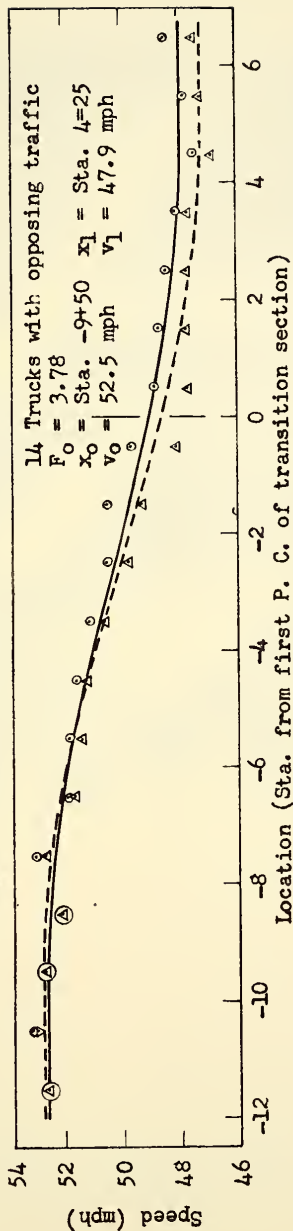
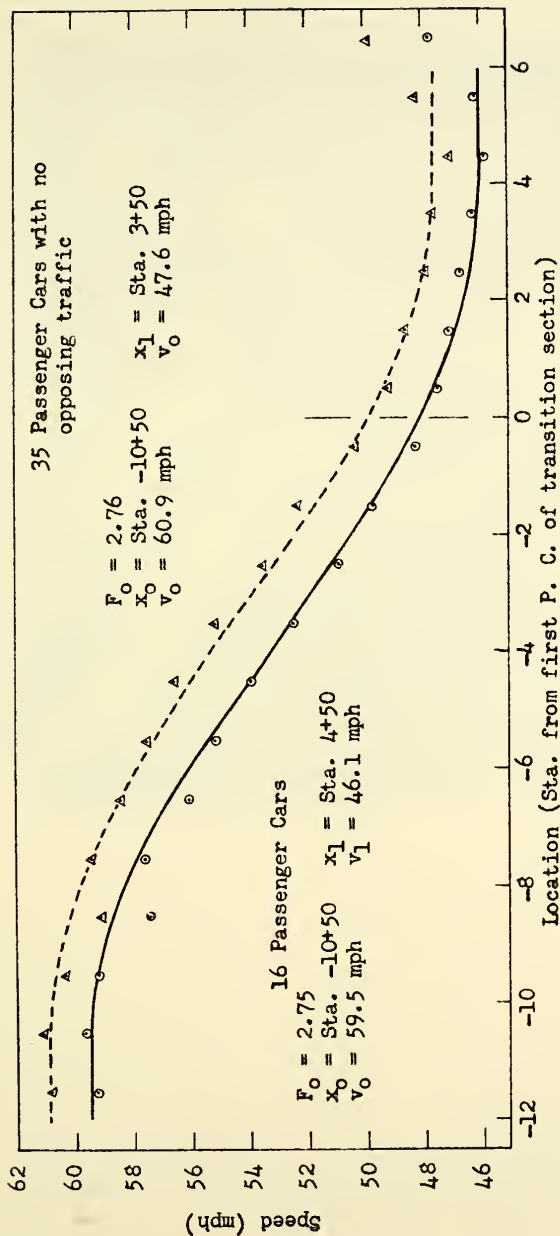


Figure 21. Speed-Location Graph for Transition from 4-Lanes to 2-Lanes on U. S. 52 near Templeton, Indiana.

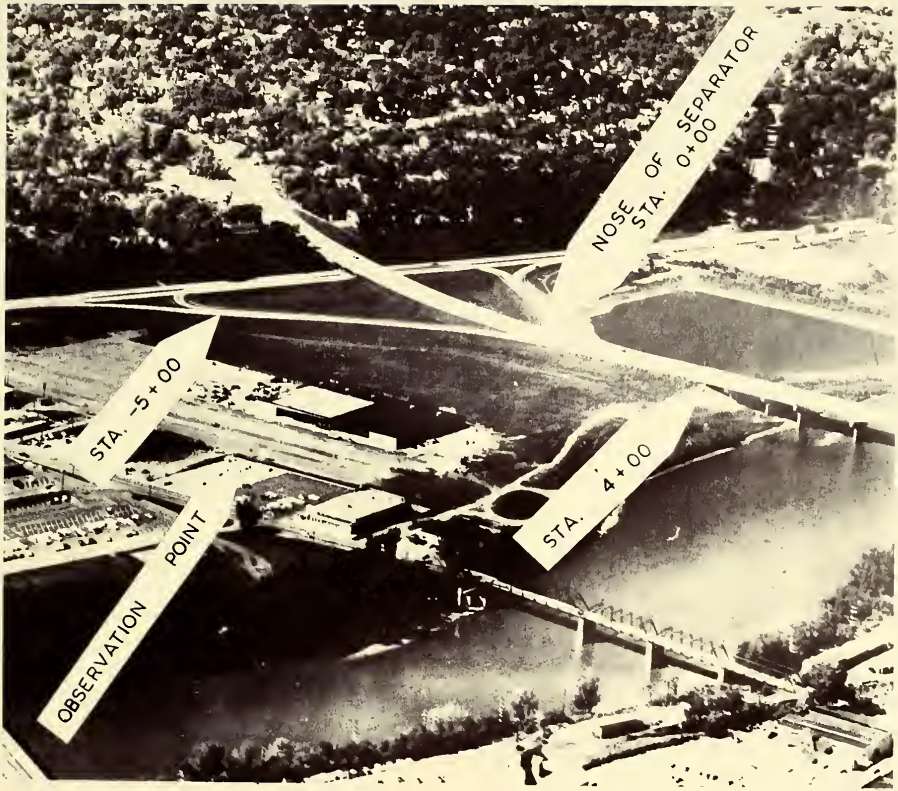


FIGURE 22. SITE OF MERGING SECTION ON
APPROACH TO WM. HARRISON BRIDGE IN WEST
LAFAYETTE, INDIANA

The average observed speeds and the frequency of calculated factors are listed in Tables A26 and A27 respectfully.

Results. Although the entire speed-location graph (Fig. 23) is rather complex, it was easily broken into individual parts for study. The speed-location graph for trucks is shown but due to the small number of such vehicles (only 5) in evidence during the time of observation no conclusions were drawn from these data.

The theoretical curve again closely approximates the observed speeds for the merging section thus indicating an excellent fit of the theoretical curve and the observed data. Here also the speed change distribution plots as a straight line on probability paper. The modulus of geometric aspects for the merging area was found to be 2.81 from the observed data.

Narrow Bridge

Indiana 43 near Chalmers

Description. The bridge under consideration is an open truss bridge with an interior width of 22 feet 4 inches which is the same as the width of pavement on the approach to the facility (Fig. 24). The approach is level and straight. Visibility is not restricted. A sign warning of a narrow bridge is located about 600 feet prior to the bridge. A record of the observed speeds is in Table A28.

Results. There was only a minor speed change observed on the approach to the narrow bridge. This small speed reduction was statistically insignificant and thus the other calculations would have little basis for meaning. The only estimate of F_0 would be a high value indicating no resistance offered by the highway feature.

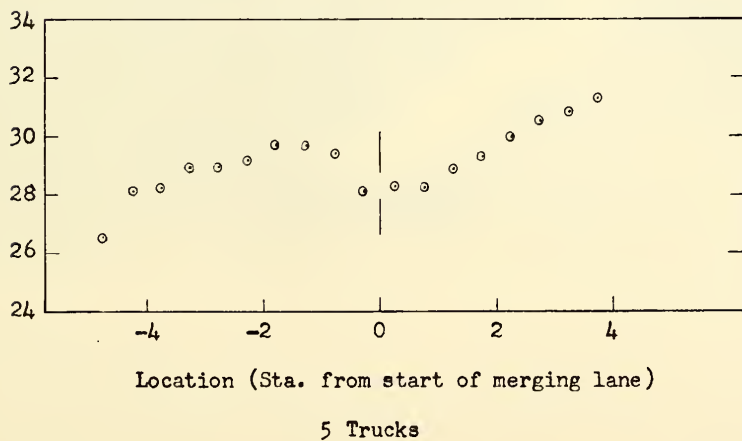
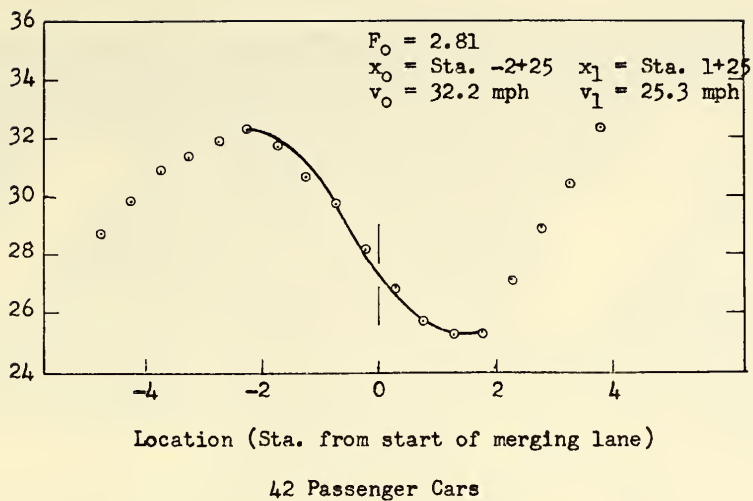


Figure 23. Speed-Location Graph for Merging Section of Approach to Wm. Harrison Bridge in West Lafayette, Indiana.

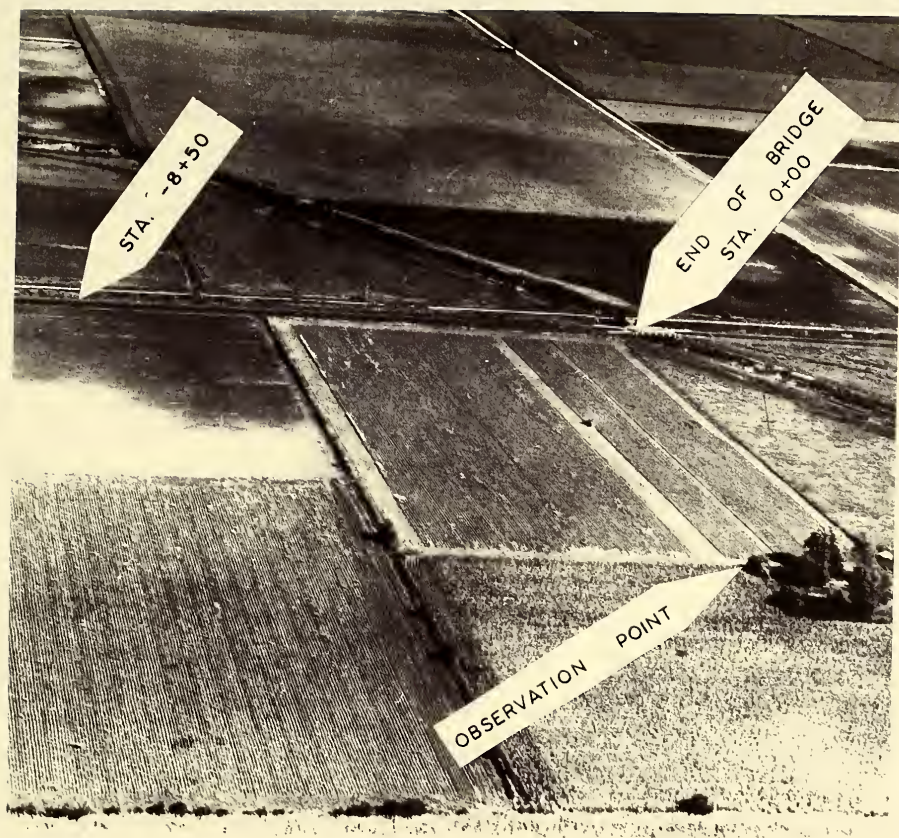


FIGURE 24. SITE OF NARROW BRIDGE
ON INDIANA 43 NEAR CHALMERS, INDIANA

Comments. The results on speed-change of these observations are in contrast with the results published by Quimby (54). Quimby's work was concerned with the effect on vehicle approach speeds of various signing and pavement markings located prior to a similar but narrower bridge. He found considerable reduction in vehicle speeds in the approach to the narrow truss bridge which he observed. His data were not available for analysis in the manner established in this study.

CONCLUSIONS

1. (a) The pressure potential theory provides an excellent theoretical model for explaining the variations observed in speeds on the approach to and through most geometric highway features. It does not apply to geometric features requiring severe speed changes.
(b) The modulus of geometric aspects provides a reproducible quantitative rating of the ease of traffic flow through all geometric features except those requiring severe speed changes.
2. (a) Vehicle speeds for both the open highway condition and in the vicinity of geometric highway features appear to be stochastic.
(b) On the basis of the observations of the effects of ambient conditions during the conduct of testing at the pilot section, it is suspected that the speed change distribution for individual vehicles at a particular geometric highway feature has a constant mean value for all ambient conditions.
3. The photographic method of data collection, although restricted to open areas and requiring expensive equipment, provides an adequate coverage of data for the type of study undertaken herein.

RECOMMENDATIONS

The success of the modulus of geometric aspects as a quantitative rating of the highway features considered in this study suggests that the following additional investigations be undertaken.

1. A systematic tabulation of the modulus of geometric aspects for all pertinent highway features should be initiated. It is hoped that a spectrum of these values would: (a) allow design decisions to be made in a quantitative manner; and (b) provide a basis for simulation approaches to traffic flow.
2. The photographic method of data collection should be reviewed for the purpose of increasing the method's efficiency and use.
3. The relationship between the modulus of geometric aspects for cars and that for trucks should be explored in an attempt to obtain an equivalency between these vehicle types.
4. The theory should be amplified to include the compound modulus of geometric aspects as revealed in highway features requiring severe speed changes.

SELECTED LIST OF REFERENCES

SELECTED LIST OF REFERENCES

1. _____ "A Policy on Design Standards for the National System of Interstate and Defence Highways," American Association of State Highway Officials, 1956.
2. Gerlough, D. L. and J. H. Mathewson, "Approaches to Operational Problems in Street and Highway Traffic," Journal Operations Research, Vol. 4, pp. 32-41, 1956.
3. Greenshields, B. D. and F. M. Weida, "Statistics with Applications to Highway Traffic Analyses," The Eno Foundation for Highway Traffic Control, 1952.
4. Miller, A. J., "Traffic Flow Treated as a Stochastic Process," In Theory of Traffic Flow, R. Herman, Editor, Elsevier Publishing Co., pp. 165-174, 1961.
5. Williams, T., "Vehicle Speeds and Delays," Engineering, Vol. 186, No. 4831, pp. 480-82, 1958.
6. _____ "Analysis of Speed Study Data," Michigan State Highway Department, C. M. Ziegler, State Highway Commissioner, 1955.
7. Tharp, K. J. and D. F. Petty, "Traffic Speed Report No. 74," Joint Highway Research Project, Purdue University, 1961.
8. Hurd, F., "Highway Design, Traffic Control and Driving," in "The Motor-Vehicle Driver: His Nature and Improvement," R. C. F. Goetz, Ed., The Eno Foundation for Highway Traffic Control, pp. 80-99, 1949.
9. Avery, E. V., "Effect of Raising Speed Limits on Urban Arterial Streets," Highway Research Board Bulletin 244, pp. 88-97, 1960.
10. Harr, M. E. and G. A. Leonards, "A Theory of Traffic Flow for Evaluation of Geometric Aspects of Highways," Highway Research Board Bulletin 308, pp. 87-91, 1962.
11. Wardrop, J. G., "Some Theoretical Aspects of Road Traffic Research," Proceedings, Institute of Civil Engineers, Vol. 1, pp. 325-362, 1952.
12. Haight, F. A., "Mathematical Theories of Road Traffic," Special Report, Institute of Transportation and Traffic Engineering, University of California, 1960.

13. _____ "Highway Capacity Manual," U. S. Department of Commerce, Bureau of Public Roads, 1950.
14. Pipes, L. A., "A Proposed Dynamic Analogy of Traffic," Journal of Applied Physics, Vol. 24, pp. 274-286, 1953.
15. Chandler, R., R. Herman, and E. Montroll, "Traffic Dynamics: Studies in Car Following," Operations Research, Vol. 6, pp. 165-184, 1958.
16. Kometani, E. and T. Sasaki, "A Safety Index for Traffic with Linear Spacing," Operations Research, Vol. 7, pp. 704-720, 1959.
17. Herman, R., E. W. Montroll, R. B. Potts, and R. W. Rothery, "Traffic Dynamics: Analysis of Stability in Car-Following," Operations Research, Vol. 7, pp. 86-106, 1959.
18. Newell, G., "A Theory of Platoon Formation in Tunnel Traffic," Operations Research, Vol. 7, pp. 589-598, 1959.
19. Olcott, E. S., "The Influence of Vehicular Speed and Spacing on Tunnel Capacity," Journal of the Operations Research Society of America, Vol. 3, pp. 146-167, 1955.
20. Lighthill, M. and G. Whitham, "On Kinematic Waves; II, A Theory of Traffic Flow on Long Crowded Roads," Proceedings of the Royal Society A, Vol. 229, pp. 317-345, 1955.
21. Greenburg, H., "An Analysis of Traffic Flow," Operations Research, Vol. 7, pp. 79-86, 1959.
22. Newell, G. "Mathematical Models for Free-Flowing Highway Traffic," Journal of the Operations Research Society of America, Vol. 3, pp. 176-186, 1955.
23. Richards, P. I., "Shock Waves on the Highway," Operations Research, Vol. 4, pp. 42-51, 1956.
24. Edie, L. and R. Foote, "Effect of Shock Waves on Tunnel Traffic Flow," Proceedings, Highway Research Board, Vol. 39, pp. 492-505, 1960.
25. Edie, L. and R. Foote, "Traffic Flow in Tunnels," Proceedings, Highway Research Board, Vol. 37, pp. 334-344, 1958.
26. Edie, L. C., "Car-Following and Steady-State Theory for Non-Congested Traffic," Operations Research, Vol. 9, pp. 66-76, 1961.
27. Gazis, D., R. Herman, and R. Potts, "Car-Following Theory of Steady-State Traffic Flow," Operations Research, Vol. 7, pp. 499-505, 1959.
28. Wohl, M., "Simulation - Its Application to Traffic Engineering," Traffic Engineering, Vol. 30, No. 11, pp. 13-17, 29, 1960.

29. Helly, W., "Simulation of Bottlenecks in Single Lane Traffic Flow," in Theory of Traffic Flow, R. Herman, Ed., Elsevier Publishing Company, pp. 205-238, 1961.
30. Mathewson, J. D., D. L. Trautman, and D. L. Gerlough, "Study of Traffic Flow by Simulation," Proceedings, Highway Research Board, Vol. 34, pp. 522-530, 1955.
31. Goode, H. H., C. H. Pollmar, and J. B. Wright, "The Use of a Digital Computer to Model a Signalized Intersection," Proceedings, Highway Research Board, Vol. 35, pp. 548-557, 1956.
32. Gerlough, D., "Use of Poisson Distribution in Highway Traffic," in Poisson and Traffic, Eno Foundation for Highway Traffic Control, pp. 1-58, 1955.
33. Schuhl, A., "The Calculation of Probabilities and the Distribution of Traffic on Two Lane Roads," in Poisson and Traffic, Eno Foundation for Highway Traffic Control, pp. 59-75, 1955.
34. Adams, W. F., "Road Traffic Considered as a Random Series," Journal, Institute of Civil Engineers, Vol. 4, pp. 121-130, 1936.
35. Lewis, R. M., "The Simulation of Traffic Flow to Obtain Volume Warrants for Intersection Control," Unpublished Ph. D. Thesis, Purdue University, 1962.
36. Webster, F. V., "Delays at Traffic Signals: Fixed-time Signals," Road Research Note 2374, Great Britain, Road Research Laboratory, 1955.
37. May, A., "A Friction Concept of Traffic Flow," Proceedings, Highway Research Board, Vol. 38, pp. 493-510, 1959.
38. Miller, A. J., "A Queueing Model for Road Traffic Flow," Journal of the Royal Statistical Society (B), pp. 64-75, 1961.
39. Charnes, A. and W. W. Cooper, "Multicopy Traffic Network Models," in Theory of Traffic Flow, R. Herman, Ed., pp. 85-96, 1961.
40. Greenshields, B. D., "A Study of Traffic Capacity," Proceedings, Highway Research Board, Vol. 14, pp. 448-477, 1934.
41. Smeed, R. J., "Theoretical Studies and Operational Research on Traffic and Traffic Congestion," Bulletin Institute International de Statistique (Stockholm, Sweden), Vol. 36, No. 4, pp. 347-375, 1958.
42. Malo, A., H. Mika, and V. Walbridge, "Traffic Behavior on an Urban Expressway," Highway Research Board Bulletin 235, pp. 19-37, 1959.
43. Taragin, A., "Driver Performance on Horizontal Curves," Public Roads, Vol. 28, No. 2, pp. 27-39, 1954.

44. Oppenlander, J., "Multivariate Analysis of Vehicular Speeds," Unpublished thesis, University of Illinois, 1962.
45. Willey, W., "Survey of Uphill Speeds of Trucks on Mountain Grades," Proceedings, Highway Research Board, Vol. 29, pp. 304-310, 1949.
46. Taragin, A., "Effect of Length of Grade on Speed of Motor Vehicles," Proceedings, Highway Research Board, Vol. 25, pp. 342-353, 1945.
47. Saal, C. C., "Hill-Climbing Ability of Motor Trucks," Proceedings, Highway Research Board, Vol. 21, pp. 393-407, 1941.
48. Lefevre, B., "Speed Characteristics on Vertical Curves," Proceedings, Highway Research Board, Vol. 32, pp. 395-413, 1953.
49. Greenshields, B. D., "The Quality of Traffic Flow," in Quality and Theory of Traffic Flow, Bureau of Highway Traffic, Yale University, pp. 1-40, 1957.
50. Breuning, S. and A. Bone, "Direct Evaluation of Geometric Highway Design," Proceedings, Highway Research Board, Vol. 37, pp. 358-368, 1958.
51. Dixon, W. J. and F. J. Massey, "Introduction to Statistical Analysis," McGraw-Hill Book Company, Inc., Second Edition, 1957.
52. Matson, T. M., W. S. Smith, and F. W. Hurd, Traffic Engineering, McGraw-Hill Book Company, Inc., 1955.
53. Beaky, J., "Acceleration and Deceleration Characteristics of Private Passenger Vehicles," Proceedings, Highway Research Board, Vol. 18, pp. 85-88, 1938.
54. Quimby, W. S., "Traffic Patterns at a Narrow Bridge," Proceedings, Highway Research Board, Vol. 27, pp. 281-290, 1947.

APPENDIX I

NOMENCLATURE

p = Motivating pressure potential.

x = A specific location along the roadway.

L = A particular length of roadway (feet).

v = Vehicle speed (miles per hour).

t = Time (seconds).

ρ = Vehicle density (vehicles per unit of effective area).

w = Effective width of roadway.

N = Number of vehicles.

c_1, c_2, k = Coefficients of proportionality.

$a^2 = (c_1 + c_2)(k)^{-1} = \text{Constant.}$

P = Passenger car.

T = Truck

P.C. = Point of curvature, that is the location where the vehicle first encounters the actual change in direction in a horizontal highway curve. The station of the P.C. in all cases was taken as 0+00. The stations of all points was taken as positive or negative relative to the P.C. For example, a point 243 feet on the curve would be denoted as Sta. 2+43, whereas a point 243 feet prior to the P.C. would be Sta. -2+43.

APPENDIX II

TABLE A1
VELOCITY RELATIONSHIP ($\frac{v(x,t)}{v_{avg}}$) FOR SELECTED F_0 VALUES

$\frac{v(x,t)}{v_{avg}} = 1 + 2 \sum_{n=1}^{\infty} e^{-n^2 F_0} \cos \frac{n\pi x}{L}$					
x/L	$F_0 = 0.1$	$F_0 = 0.2$	$F_0 = 0.3$	$F_0 = 0.4$	$F_0 = 0.5$
0.0	5.6048	3.9633	2.2360	2.8024	2.5066
0.1	4.3795	3.5033	2.9805	2.6348	2.3859
0.2	2.0889	2.4195	2.3288	2.1897	2.0576
0.3	0.6084	1.3057	1.5436	1.6085	1.6077
0.4	0.1080	0.5505	0.8679	1.0445	1.1381
0.5	0.0118	0.1813	0.4140	0.5995	0.7299
0.6	0.0006	0.0466	0.1675	0.3041	0.4243
0.7	0.0001	0.0093	0.0575	0.1364	0.2239
0.8	0.0000	0.0014	0.0167	0.0544	0.1085
0.9	0.0000	0.0001	0.0042	0.0205	0.0524
1.0	0.0000	0.0000	0.0009	0.0062	0.0196
x/L	$F_0 = 0.6$	$F_0 = 0.8$	$F_0 = 1.0$	$F_0 = 1.25$	$F_0 = 1.5$
0.0	2.2882	1.9817	1.7726	1.5865	1.4512
0.1	2.1960	1.9215	1.7295	1.5559	1.4284
0.2	1.9412	1.7518	1.6065	1.4677	1.3626
0.3	1.5804	1.5016	1.4209	1.3326	1.2608
0.4	1.1851	1.2105	1.1975	1.1661	1.1339
0.5	0.8187	0.9185	0.9634	0.9865	0.9950
0.6	0.5214	0.6576	0.7432	0.8120	0.8581
0.7	0.3072	0.4480	0.5564	0.6591	0.7362
0.8	0.1708	0.2986	0.4162	0.5406	0.6405
0.9	0.0976	0.2104	0.3297	0.4659	0.5796
1.0	0.0749	0.1814	0.3006	0.4404	0.5587
x/L	$F_0 = 2.0$	$F_0 = 2.5$	$F_0 = 3.0$	$F_0 = 4.0$	$F_0 = 5.0$
0.0	1.2713	1.1643	1.0996	1.0366	1.0135
0.1	1.2580	1.1562	1.0947	1.0348	1.0128
0.2	1.2192	1.1328	1.0806	1.0296	1.0109
0.3	1.1589	1.0965	1.0585	1.0215	1.0079
0.4	1.0831	1.0507	1.0308	1.0113	1.0042
0.5	0.9993	0.9999	0.9999	0.9999	0.9999
0.6	0.9158	0.9492	0.9692	0.9887	0.9958
0.7	0.8407	0.9035	0.9415	0.9785	0.9921
0.8	0.7812	0.8672	0.9194	0.9704	0.9891
0.9	0.7431	0.8439	0.9053	0.9652	0.9872
1.0	0.7300	0.8359	0.9004	0.9634	0.9896

TABLE A2

ERROR INTRODUCED BY USING THE APPROXIMATE EQUATION
FOR THE VELOCITY RELATIONSHIP

$$\text{Exact} \quad \frac{v(x,t)}{v_{avg}} = 1 + 2 \sum_{n=1}^{\infty} e^{-n^2 F_0} \cos \frac{n\pi x}{L}$$

$$\text{Approx.} \quad \frac{v(x,t)}{v_{avg}} = 1 + 2 e^{-F_0} \cos \frac{n\pi x}{L}$$

$$F_0 = 0.8$$

x/L	$\frac{v(x,t)}{v_{avg}}$		Error Introduced by Approximation
	Complete Series	Approx. Solution	
0.0	1.9817	1.8987	-0.0830
0.1	1.9215	1.8547	-0.0669
0.2	1.7518	1.7270	-0.0257
0.3	1.5016	1.5282	0.0257
0.4	1.2105	1.2777	0.0669
0.5	0.9185	1.0000	0.0815
0.6	0.6576	0.7223	0.0647
0.7	0.4480	0.4718	0.0238
0.8	0.2986	0.2730	-0.0256
0.9	0.2104	0.1453	-0.0651
1.0	0.1814	0.1013	-0.0801

$$F_0 = 1.0$$

0.0	1.7726	1.7358	-0.0368
0.1	1.7295	1.6997	-0.0298
0.2	1.6065	1.5952	-0.0113
0.3	1.4209	1.4325	0.0116
0.4	1.1975	1.2274	0.0299
0.5	0.9634	1.0000	0.0366
0.6	0.7432	0.7726	0.0294
0.7	0.5564	0.5675	0.0111
0.8	0.4162	0.4048	-0.0114
0.9	0.3297	0.3003	-0.0294
1.0	0.3006	0.2642	-0.0364

TABLE A2 (continued)

	$\frac{v(x,t)}{v_{avg}}$		
x/L	Complete Series	Approx. Solution	Error Introduced by Approximation
$F_o = 1.25$			
0.0	1.5865	1.5730	-0.0135
0.1	1.5559	1.5450	-0.0109
0.2	1.4677	1.4636	-0.0041
0.3	1.3326	1.3368	0.0042
0.4	1.1661	1.1771	0.0110
0.5	0.9865	1.0000	0.0135
0.6	0.8120	0.8229	0.0109
0.7	0.6591	0.6632	0.0041
0.8	0.5406	0.5364	-0.0042
0.9	0.4659	0.4550	-0.0009
1.0	0.4404	0.4270	-0.0134
$F_o = 1.50$			
0.0	1.4512	1.4463	-0.0051
0.1	1.4284	1.4244	-0.0040
0.2	1.3626	1.3610	-0.0016
0.3	1.2608	1.2623	0.0015
0.4	1.1339	1.1379	0.0040
0.5	0.9950	1.0000	0.0050
0.6	0.8581	0.8621	0.0040
0.7	0.7362	0.7377	0.0015
0.8	0.6405	0.6390	-0.0015
0.9	0.5796	0.5756	-0.0040
1.0	0.5587	0.5539	-0.0052



TABLE A3

VEHICLE SPEED BY STATIONS ON THE APPROACH TO AND THROUGH
THE CURVE ON U. S. 24 NEAR REYNOLDS, INDIANA

Vehicle Speed (miles per hour)														
Data Set Number														
	1		2		3		4		5		6		Total	
Sta.	P (29)	T (11)	P (54)	T (17)	P (25)	T (25)	P (35)	T (15)	P (19)	T (13)	P (47)	T (7)	P (208)	T (88)
-6+75	53.68	51.13	64.71	54.14	59.74	51.22	60.79	49.48	62.42	55.12	59.49	55.42	60.55	52.39
-6+25	53.11	49.76	62.61	53.66	59.27	50.48	60.02	49.22	62.18	54.65	59.56	54.60	59.73	51.73
-5+75	54.59	50.25	62.66	53.89	58.01	50.57	59.81	49.32	61.13	54.02	60.20	55.46	59.82	51.86
-5+25	54.35	51.16	63.00	53.64	58.52	50.79	60.04	49.72	61.42	54.52	59.58	55.49	60.01	52.13
-4+75	53.79	50.47	62.78	53.47	57.44	49.94	59.26	49.03	60.41	54.03	59.98	55.57	59.46	51.59
-4+25	53.20	50.88	62.99	54.81	57.95	50.69	59.74	49.49	61.80	53.93	59.65	55.77	59.63	52.19
-3+75	53.96	51.46	63.55	55.33	57.27	50.20	58.99	48.90	60.64	55.10	59.39	54.75	59.50	52.23
-3+25	52.91	49.54	61.76	53.40	56.88	50.23	58.88	49.06	60.93	53.80	59.16	56.02	58.81	51.54
-2+75	51.82	50.19	62.42	53.69	56.28	50.16	58.57	49.06	59.84	53.21	58.49	56.33	58.45	51.60
-2+25	52.87	51.53	61.61	53.29	56.19	49.71	58.11	49.19	59.59	54.01	58.18	55.28	58.21	51.62
-1+75	51.94	51.32	61.67	54.11	55.55	49.62	57.54	48.59	59.32	53.70	57.93	55.14	57.84	51.57
-1+25	50.90	51.45	61.71	54.66	55.26	49.52	57.33	49.12	58.55	53.80	57.55	55.88	57.48	51.83
-0+75	50.31	49.58	61.53	53.27	55.28	49.38	57.11	48.75	58.06	53.95	56.96	54.70	57.14	51.15
-0+25	48.83	48.80	59.33	52.64	54.94	49.25	56.66	48.57	57.77	54.17	56.27	54.83	56.07	50.90
0+25	48.60	49.31	59.35	52.91	54.17	49.02	56.27	48.29	56.79	53.68	55.68	55.18	55.67	50.86
0+75	50.54	51.43	60.48	54.19	53.60	48.48	55.61	48.37	56.51	52.97	55.02	53.88	55.87	51.03
1+25	48.11	49.17	58.17	51.46	53.00	48.38	54.44	47.42	55.04	53.08	54.92	55.44	54.52	50.17
1+75	47.57	49.24	58.10	52.39	53.16	48.21	54.42	47.79	54.44	53.29	53.88	53.90	54.15	50.28

TABLE A4
FREQUENCY OF
SPEEDS OF INDIVIDUAL VEHICLES APPROACHING CURVE
ON U. S. 24 NEAR REYNOLDS, INDIANA

v _o (mph)	Data Set Number													
	1		2		3		4		5		6		Total	
	P	T	P	T	P	T	P	T	P	T	P	T	P	T
Be-														
low 38	1				1		1						2	1
38 40	1					4			1				2	4
40 42				1		1	1						1	2
42 44	1			2		1		1			2		3	4
44 46	1	2	1	1	1	2		2			2		5	7
46 48	2	2				3	1	4	1	2	1	1	5	12
48 50	1	2	1	1	1	3					3		6	6
50 52	1	2	2	2	1	2	1	2		3	3	1	8	12
52 54	5	1		1	2	1	3	2		2	1	3	11	10
54 56	7		3	1	3	3	4	2			4		21	6
56 58	2	1	6	2	5		3		1	3	3		20	6
58 60	2		3	3	1	2	4		3	1	3		16	6
60 62	2	1	5	1			8		3	1	4		22	3
62 64			7	1	4	2			1		7	1	19	4
64 66	1		8		1		3	1	3	1	2		18	2
66 68	1		5		1		1		2		5	1	15	1
68 70	1		4				3		1		2		11	
70 72			2		1	1					2		5	1
72 74			5	1			1				1		7	1
74 76			1		1		1		2				5	
76 & above			1		2		1				2		6	
Total	29	11	54	17	25	25	35	15	18	13	47	7	208	88
Ave. Speed	53.90	50.45	63.26	54.06	59.24	50.12	60.20	49.40	61.89	54.38	59.60	55.29	60.01	51.86
Std. Dev.	7.45	4.99	6.89	8.36	9.59	8.47	7.60	6.51	8.65	6.85	8.71	8.16	8.42	7.41

TABLE A5
FREQUENCY OF
MINIMUM SPEEDS OF INDIVIDUAL VEHICLES ON CURVE
OF U. S. 24 NEAR REYNOLDS, INDIANA

v ₁ (mph)	Data Set Number													
	1		2		3		4		5		6		Total	
	P	T	P	T	P	T	P	T	P	T	P	T	P	T
Be-														
low 34	1								1				2	
34 36	1				1								2	
36 38					1	2		1					1	3
38 40	1				1	2				1			2	3
40 42	1					2	1				6		8	2
42 44	2	2	1	1	1	2	1	3	1		3		9	8
44 46	4		1	2	1	2	5	4		1	2	1	13	10
46 48	3	1	1	1	2	3	2	1	1	1	2	1	11	8
48 50	6	4	5	2	1	4	2	2	1	1	5	1	20	14
50 52	2	1	1	3	5	3	4	3	1	3	2		15	13
52 54	3	1	6	1	2		6		5	1	5	2	27	5
54 56	2	1	8		2	3	2		1	1	4		19	5
56 58	2	1	5	3	2		6		2	2	4		21	6
58 60			9	2	3		1	1	1		8		22	3
60 62	1		3			2	2		2	1	2		10	3
62 64			4	1	2		1		1	1	1		9	2
64 66			5	1					1			2	6	3
66 68			3		1		1						5	
68 & above			2				1				3		6	
Total	29	11	54	17	25	25	35	15	18	13	47	7	208	88
Ave. Speed	48.10	49.55	57.48	53.00	51.88	47.48	53.00	47.00	53.67	52.23	52.83	53.86	53.37	49.91
Std. Dev.	6.38	4.40	6.28	6.48	8.05	6.72	6.67	5.10	7.65	6.65	6.96	8.15	7.57	6.66

TABLE A6
FREQUENCY OF
SPEED CHANGES OF INDIVIDUAL VEHICLES AT CURVE
ON U. S. 24 NEAR REYNOLDS, INDIANA

v ₀ -v ₁ (mph)		Data Set Number														
		1		2		3		4		5		6		Total		
		P	T	P	T	P	T	P	T	P	T	P	T	P	T	
-4	-3				2											2
-3	-2	2		3	2									5	2	
-2	-1			1			2					1		1	3	
-1	0	1	4	1	2		1	1	1		1	1	1	4	10	
0	1	2	1	2	3	1	4	1	3		1	2	2	8	14	
1	2	2	4	2	4	3	3		5	1	5	1		9	21	
2	3	1		3	1	3	2	4	1		3	3	1	14	8	
3	4	3	1	4		1	3		1	3	2	4	1	15	8	
4	5	4	1	5		1	4	4	2	3		6		23	7	
5	6	1		7	1		3	2	1	1		6		17	5	
6	7	5		5		4		2		1		2		19		
7	8	2		6	1	2	6					7		23	1	
8	9			4		3	1	5				3	1	15	2	
9	10			4	1	2	1	4	1	1		1		12	3	
10	11	2		1		1	1	3		2		5		14	1	
11	12			2		1		1		1	1	3		8	1	
12	13	1		1		2		1		2				7		
13	14	1		1										2		
14	15							1						1		
15	16			1						2				3		
16	17	1		1								1		3		
17	18															
18	&	1				1				1		2		5		
above																
Total		29	11	54	17	25	25	35	15	18	13	47	7	208	88	
Ave. Speed Change		5.67	1.14	5.89	1.15	6.86	3.26	7.07	2.57	8.55	2.58	6.90	1.93	6.63	2.26	
Std. Dev.		5.14	1.69	4.25	3.60	4.38	3.17	3.39	2.58	5.04	2.89	4.24	3.36	4.36	3.04	

TABLE A7
FREQUENCY OF
RATIOS OF SPEED CHANGES TO APPROACH SPEEDS
FOR INDIVIDUAL VEHICLES FOR CURVE ON U. S. 24 NEAR REYNOLDS, INDIANA

$\frac{v_o-v_1}{v_o}$		Data Set Number														Total	
		1		2		3		4		5		6					
		P	T	P	T	P	T	P	T	P	T	P	T				
-.10	-.08				1											1	
-.08	-.06	1		1	1							1		3		1	
-.06	-.04			1	2									1		2	
-.04	-.02			2		1						1	1	4		1	
-.02	.00	1	5	1	2		1	2			1		1	4	10		
.00	.02	2	1	2	5		6	1	4		2		2	7	20		
.02	.04	4	2	5	3	3	2	1	4	1	4	1	1	15	16		
.04	.06	2		3		2	3	3	2	3	5	2		15	10		
.06	.08	3	3	5		3	5	4	1	3		11	1	29	10		
.08	.10	3		12			2	3	3	1		5		24	5		
.10	.12	6		4	1	4	1	6		1		5		26	2		
.12	.14	2		6	2	4	3	3				5		20	5		
.14	.16	1		6		3	1	3				5		18	1		
.16	.18			2		1	1	4		1		4	1	12	2		
.18	.20	1		1		3		3	1	4				12	1		
.20	.22			3				2		1	1	2		8	1		
.22	.24	1								1		1		3			
.24	.26	1								1				2			
.26	&	1				1				1		2		5			
above																	
Total		29	11	54	17	25	25	35	15	18	13	47	7	208	88		
Ave. Ratio		.0914	.0209	.0896	.0135	.1116	.0644	.1111	.0526	.1378	.0454	.1083	.0357	.1045	.0420		
Std. Dev.		.0765	.0351	.0645	.0633	.0665	.0505	.0602	.0413	.0797	.0529	.0617	.0532	.0685	.0551		

TABLE A8
FREQUENCY OF
e-F₀ FACTORS FOR INDIVIDUAL VEHICLES TRAVERSING CURVE
ON U. S. 24 NEAR REYNOLDS, INDIANA

e^{-F_0}		Data Set Number															
		1		2		3		4		5		6		Total			
		P	T	P	T	P	T	P	T	P	T	P	T	P	T		
Be-																	
low	-.02			1												1	
-.02	-.01	2		3										4		3	
-.01	.00	1	4	3	2		3	1	1			2	2	7		13	
.00	.01	4	5	7	7	4	6	2	7	1	6	4	3	22		34	
.01	.02	5	2	6	1	5	6	6	3	6	5	11	1	39		18	
.02	.03	7		16	1	2	5	7	3	2		9		43		9	
.03	.04	4		10	2	8	3	7				9		38		5	
.04	.05	1		7		3	2	8		1		7	1	27		3	
.05	.06	1		3		2		4	1	5		2		17		1	
.06	.07	1								1	1			2		1	
.07	.08	1								1		3		5			
.08	.09	2												2			
.09 & above						1				1				2			
Total		29	11	54	17	25	25	35	15	18	13	47	7	208		88	
Ave. e^{-F_0}		.0271	.0032	.0243	.0038	.0306	.0170	.0313	.0137	.0389	.0129	.0290	.0093	.0289		.0109	
Std. Dev.		.0262	.0075	.0173	.0169	.0204	.0147	.0158	.0146	.0259	.0169	.0191	.0172	.0202		.0156	

TABLE A9
FREQUENCY OF
F₀ VALUES FOR INDIVIDUAL VEHICLES TRAVERSING CURVE
ON U. S. 24 NEAR REYNOLDS, INDIANA

F ₀		Data Set Number														Total	
		1		2		3		4		5		6					
		P	T	P	T	P	T	P	T	P	T	P	T	P	T		
2.0	2.5	2				1				1				4			
2.5	3.0	3		4		3		4	1	7	1	5		26		2	
3.0	3.5	5		16	2	10	5	14		1		16	1	62		8	
3.5	4.0	8	2	18	2	4	5	10	3	5		11	1	56		13	
4.0	4.5	3		4		2	6	2	1	3	4	9		23		11	
4.5	5.0	3	2	3	1	2		3	5		1	2	1	13		10	
5.0	5.5		3	2	4	2	2	1	2	1	5			6		16	
5.5	6.0	1			1	1	1						1	2		3	
6.0	6.5	1		2					1					3		1	
6.5	7.0			1			1					1		2		1	
7.0	+ ∞				1		2		1		1	1	1	1		6	

- ∞	-6.0		1	1	2		1	1			1		1	2		6	
-6.0	-5.5	1	1						1					1		2	
-5.5	-5.0		2				2									4	
-5.0	-4.5											1	1	1		1	
-4.5	-4.0	2		3	3									5		3	
-4.0	-3.5				1							1		1		1	
Total		29	11	54	17	25	25	35	15	18	13	47	7	208		88	

*** A negative value of F₀ indicates that the vehicle increased speed through the feature.

TABLE A10

VEHICLE SPEED BY STATIONS ON THE APPROACH TO AND THROUGH
THE CURVE ON U. S. 41 (and 52) NEAR EARL PARK, INDIANA

Sta.	Vehicle Speed (miles per hour)			
	Opposing Lane			
	Occupied		Free	
	P (38)	T (23)	P (48)	T (21)
-8+75	53.42	49.63	53.06	48.79
-8+25	53.24	49.65	52.95	48.58
-7+75	52.95	49.49	52.63	48.61
-7+25	52.44	49.32	52.28	48.47
-6+75	52.30	49.46	52.28	48.61
-6+25	52.47	49.61	52.37	48.65
-5+75	51.95	49.05	51.88	48.32
-5+25	51.01	48.92	50.88	48.03
-4+75	50.10	48.61	50.20	47.62
-4+25	49.94	48.51	49.98	47.37
-3+75	49.50	48.19	49.53	47.28
-3+25	48.62	47.71	48.74	46.78
-2+75	47.77	47.12	48.11	46.18
-2+25	46.52	46.21	47.16	45.72
-1+75	45.42	45.49	46.04	45.23
-1+25	45.73	46.55	46.75	46.24
-0+75	43.44	44.71	44.53	44.80
-0+25	41.78	43.67	42.90	43.86
0+25	41.13	43.25	42.31	43.60
0+75	40.82	43.00	41.95	43.70
1+25	40.72	42.83	41.77	43.51
1+75	41.04	42.98	42.05	43.74
2+25	41.38	42.86	42.14	43.79
2+75	42.06	43.10	42.79	44.10
3+25	42.17	42.71	42.96	43.95
3+75	43.54	43.26	43.96	44.43

TABLE A11
FREQUENCY OF
SPEEDS OF INDIVIDUAL VEHICLES APPROACHING CURVE
ON U. S. 41 (and 52) NEAR EARL PARK, INDIANA

v _o (mph)	Opposing Lane				Total	
	Free		Occupied		Total	
	P	T	P	T	P	T
Below 38	1	1			1	1
38 40	2	1		1	2	2
40 42	2				2	
42 44	1		1	2	2	2
44 46	1	1	3	2	4	3
46 48	6	5	2	3	8	8
48 50	2	4	7	6	9	10
50 52	5	6	4	1	9	7
52 54	5	2	5	4	10	6
54 56	6		4		10	
56 58	6		3	3	9	3
58 60	4		3		7	
60 62	1		2		3	
62 64	3	1	3	1	6	2
64 66	2				2	
66 Above	1		1		2	
Total	48	21	38	23	86	44
Ave. Speed	52.92	48.90	53.42	49.87	53.14	49.41
Std. Dev.	7.26	5.16	5.81	5.46	6.66	5.28

TABLE A12
FREQUENCY OF
MINIMUM SPEEDS OF INDIVIDUAL VEHICLES ON CURVE
OF U. S. 41 (and 52) NEAR EARL PARK, INDIANA

v ₁ (mph)	Opposing Lane					
	Free		Occupied		Total	
	P	T	P	T	P	T
Below 32	1	1	1		2	1
32 34	1			1	1	1
34 36	7		4		11	
36 38	4		6	2	10	2
38 40	5	4	8	4	13	8
40 42	4	3	4	1	8	4
42 44	4	5	4	6	8	11
44 46	9	3	5	3	14	6
46 48	8	2	4	4	12	6
48 50	4	2	1		5	2
50 52			1	2	1	2
52 Above	1	1			1	1
Total	48	21	38	23	86	44
Ave. Speed	42.04	43.10	40.89	42.91	41.53	43.00
Std. Dev.	5.33	4.56	4.58	4.49	4.97	4.52

TABLE A13
FREQUENCY OF
SPEED CHANGES OF INDIVIDUAL VEHICLES AT CURVE
ON U. S. 41 (and 52) NEAR EARL PARK, INDIANA

v ₀ -v ₁ (mph)		Opposing Lane				Total	
		Free		Occupied			
		P	T	P	T	P	T
Below	1		3		1		4
1	2		1				1
2	3	4	2		1	4	3
3	4		2		3		5
4	5	3	1	2	4	5	5
5	6	2	3	1	2	3	5
6	7		2	1	4	1	6
7	8	3	2	3	1	6	3
8	9	4	3	2	2	6	5
9	10	5	1	1	2	6	3
10	11	2		2	1	4	1
11	12	3		5		8	
12	13	3	1	3		6	1
13	14	5		6		11	
14	15	4		1		5	
15	16			4	1	4	1
16	17	5		2		7	
17	18	2			1	2	1
18	19	1		1		2	
19	20			2		2	
20	21	1		1		2	
21	Above	1		1		2	
Total		48	21	38	23	86	44
Ave. Speed Change		11.15	5.31	12.53	6.76	11.76	6.06
Std. Dev.		4.90	3.31	4.35	3.93	4.69	3.67

TABLE A14
FREQUENCY OF
RATIO OF SPEED CHANGES TO APPROACH SPEEDS FOR INDIVIDUAL
VEHICLES FOR CURVE ON U. S. 41 (and 52) NEAR EARL PARK, INDIANA

$\frac{v_0 - v_1}{v_0}$	Opposing Lane					
	Free		Occupied		Total	
	P	T	P	T	P	T
Below .02		3		1		4
.02 .04		1				1
.04 .06	1	2		1	1	3
.06 .08	3	1		2	3	3
.08 .10		1	1	5	1	6
.10 .12	3	4	2	3	5	7
.12 .14	4	1	2	4	6	5
.14 .16	5	4	1		6	4
.16 .18	4	2	2	3	6	5
.18 .20	3	1	1	1	4	2
.20 .22	3		6		9	
.22 .24	7	1	6		13	1
.24 .26	3		4	1	7	1
.26 .28	5		4		9	
.28 .30	2		5	2	7	2
.30 .32	2				2	
.32 .34	2		2		4	
.34 Above	1		2		3	
Total	48	21	38	23	86	44
Ave. Ratio	.201	.107	.231	.132	.214	.120
Std. Dev.	.076	.063	.066	.071	.073	.068

TABLE A15
FREQUENCY OF
 e^{-F_0} FACTORS FOR INDIVIDUAL VEHICLES TRAVERSING CURVE
ON U. S. 41 (and 52) NEAR EARL PARK, INDIANA

e^{-F_0}		Opposing Lane				Total	
		Free		Occupied			
		P	T	P	T	P	T
-.02	-.01		1				1
-.01	.00		1		1		2
.00	.01		2				2
.01	.02	4	3		3	4	6
.02	.03	1	4	2	8	3	12
.03	.04	8	4	3	4	11	8
.04	.05	7	5	3	3	10	8
.05	.06	6		5	1	11	1
.06	.07	8	1	10	1	18	2
.07	.08	7		6		13	
.08	.09	2		5	2	7	2
.09	.10	3		1		4	
.10	.11	1		1		2	
.11	.12			1		1	
.12	Above	1		1		2	
Total		48	21	38	23	86	44
Avg. e^{-F_0}		.057	.027	.067	.035	.061	.031
Std. Dev.		.078	.061	.071	.068	.076	.065

TABLE A16
 FREQUENCY OF
 F_0 VALUES FOR INDIVIDUAL VEHICLES TRAVERSING CURVE
 ON U. S. 41 (and 52) NEAR EARL PARK, INDIANA

F_0		Opposing Lane					
		Free		Occupied		Total	
		P	T	P	T	P	T
2.0	2.5	7		8	2	15	2
2.5	3.0	21	1	22	2	44	3
3.0	3.5	15	8	6	7	21	15
3.5	4.0	1	5	2	9	10	14
4.0	4.5	4	3		2	8	5
4.5	5.0		1				1

-6.5	-6.0		2		1		3
-6.0	-5.5						
-5.5	-5.0						
-5.0	-4.5		1				1
Total		48	21	38	23	86	44

*** A negative value of F_0 indicates that the vehicle increased speed through the feature.

TABLE A17

VEHICLE SPEEDS ON THE APPROACH TO AND THROUGH
THE CORNER ON INDIANA 26 NEAR PINE VILLAGE, INDIANA

Sta.	Vehicle Speed (miles per hour)	
	P (36)	T (18)
-9+25	47.53	42.11
-8+75	47.58	42.43
-8+25	47.05	41.83
-7+75	47.31	41.99
-7+25	46.89	41.57
-6+75	46.71	41.16
-6+25	46.11	40.56
-5+75	46.16	40.58
-5+25	45.07	39.82
-4+75	44.06	38.84
-4+25	43.30	38.44
-3+75	42.71	37.62
-3+25	41.62	36.71
-2+75	40.63	35.84
-2+25	38.83	34.71
-1+75	36.75	33.36
-1+25	33.43	30.78
-0+75	30.04	28.27
-0+25	25.25	24.83
0+25	20.05	21.29
0+75	16.89	18.01
1+25	17.80	17.90

TABLE A18

FREQUENCY DISTRIBUTION OF FACTORS (v_0 , v_1 , $v_0 - v_1$, $\frac{v_0 - v_1}{v_0}$, e^{-F_0} , F_0)

FOR CORNER ON INDIANA 26 NEAR PINE VILLAGE, INDIANA

v_0 mph	Frequency		v_1 mph	Frequency		$v_0 - v_1$ mph	Frequency	
	P	T		P	T		P	T
Below 32		2	13 14	3		12 14		2
32 34	2	2	14 15	1		14 16		1
34 36	2		15 16	3	2	16 18	2	1
36 38		1	16 17	6	4	18 20	2	1
38 40	2	3	17 18	12	3	20 22		2
40 42	1		18 19	5	4	22 23	3	3
42 44	3	3	19 20	2	4	24 26	2	1
44 46	2		20 21	2	1	26 28	2	2
46 48	3	2	21 22	1		28 30	2	
48 50	4	1	22 23	1		30 32	9	1
50 52	9	2				32 34	6	2
52 54	1	1				34 36	1	1
54 56	3	1				36 38	3	1
56 58	3					38 40	2	
Above 58	1					40 42	2	
Total	36	18		36	18		36	18

TABLE A18 (continued)

$\frac{v_0 - v_1}{v_0}$		Frequency		$e^{-F_0^*}$		Frequency		F_0^*		Frequency	
		P	T			P	T			P	T
Below 0.44			1	Below 0.14			1	Below 1.30		2	
0.44 0.46			1	0.14 0.15			1	1.30 1.35		5	
0.46 0.48			1	0.15 0.16			1	1.35 1.40		3	1
0.48 0.50	2			0.16 0.17	2			1.40 1.45		8	2
0.50 0.52			1	0.17 0.18			2	1.45 1.50		6	2
0.52 0.54	1		3	0.18 0.19	3		4	1.50 1.55		2	2
0.54 0.56	1		2	0.19 0.20	1		1	1.55 1.60		4	1
0.56 0.58	2		1	0.20 0.21	4			1.60 1.65		1	1
0.58 0.60	3		1	0.21 0.22	2		2	1.65 1.70		2	3
0.60 0.62	3		2	0.22 0.23	3		1	1.70 1.75		1	2
0.62 0.64	6		2	0.23 0.24	6		2	1.75 1.80			1
0.64 0.66	7		2	0.24 0.25	6		3	1.80 1.85		2	
0.66 0.68	3		1	0.25 0.26	3			1.85 1.90			1
0.68 0.70	4			0.26 0.27	3			1.90 1.95			1
0.70 0.72	2			0.27 0.28	1			Above 1.95			1
0.72 0.74	2			0.28 0.29	1						
				0.29 0.30	1						
Total		36	18			36	18			36	18

* Calculated by equation 12.

TABLE A19

VEHICLE SPEED BY STATIONS FOR THE 4-LANE TO 2-LANE
TRANSITION ON U. S. 52 NEAR TEMPLETON, INDIANA

Sta.	Vehicle Speed (miles per hour)			
	Opposing Lane			
	Occupied		Free	
	P (15)	T (14)	P (35)	T (18)
-11+50	59.32	52.55	60.80	52.52
-10+50	59.70	53.01	61.05	52.96
- 9+50	59.13	52.57	60.34	52.66
- 8+50	57.42	52.02	59.07	52.12
- 7+50	57.55	52.95	59.50	52.61
- 6+50	56.07	51.83	58.40	51.68
- 5+50	55.12	51.76	57.55	51.35
- 4+50	53.93	51.61	56.59	51.29
- 3+50	52.49	51.12	55.13	50.56
- 2+50	50.96	50.52	53.58	49.78
- 1+50	49.72	50.52	52.32	49.33
- 0+50	48.28	49.67	50.40	48.19
0+50	47.44	48.92	49.21	47.79
1+50	47.05	48.72	48.60	47.76
2+50	46.64	48.43	47.94	47.80
3+50	46.28	48.10	47.63	47.65
4+50	45.86	47.53	47.00	46.92
5+50	46.15	47.82	48.22	47.31
6+50	47.78	48.46	49.87	47.55

TABLE A20
FREQUENCY OF
SPEEDS OF INDIVIDUAL VEHICLES APPROACHING TRANSITION
FROM 4-LANES TO 2-LANES ON U. S. 52 NEAR TEMPLETON, INDIANA

v _o (mph)	Opposing Lane					
	Free		Occupied		Total	
	P	T	P	T	P	T
Below 46		1		1		2
46 48	1			1	1	1
48 50	1	4		2	1	6
50 52	2	4	1	4	3	8
52 54	1	2		3	1	5
54 56	1		4		5	
56 58	3	2	3		6	2
58 60	7	4	1	1	8	5
60 62	3	1			3	1
62 64	6		2	1	8	1
64 66	3		2	1	5	1
66 68	4				4	
68 70	3		2		5	
Total	35	18	15	14	50	32
Ave. Speed	60.60	53.56	59.66	52.86	60.32	53.19
Std. Dev.	5.74	3.12	5.59	5.74	5.65	5.12

TABLE A21
FREQUENCY OF
MINIMUM SPEEDS OF INDIVIDUAL VEHICLES ON TRANSITION
FROM 4-LANES TO 2-LANES ON U. S. 52 NEAR TEMPLETON, INDIANA

v ₁ (mph)	Opposing Lane					
	Free		Occupied		Total	
	P	T	P	T	P	T
Below 34	1	1			1	1
34 36	2				2	
36 38	2	1			2	1
38 40	1	1	1	2	2	3
40 42	3		3		6	
42 44	5		1	2	6	2
44 46	1	1	2	2	3	3
46 48	6	4	4	1	10	5
48 50	1	2		3	1	5
50 52	1	5	1	1	2	6
52 54	1	2	2	1	3	3
54 56	1		1		2	
56 58	4			1	4	1
58 60	4	1			4	1
60 62	1			1	1	1
62 Above	1				1	
Total	35	18	15	14	50	32
Ave. Speed	47.91	47.78	46.33	47.86	47.44	47.81
Std. Dev.	8.50	6.21	4.94	6.31	7.59	6.16

TABLE A22
FREQUENCY OF
SPEED CHANGES OF INDIVIDUAL VEHICLES DURING TRANSITION
FROM 4-LANES TO 2-LANES ON U. S. 52 NEAR TEMPLETON, INDIANA

v ₀ -v ₁ (mph)		Opposing Lane				Total	
		Free		Occupied			
		P	T	P	T	P	T
0	2		7		5		12
2	4	3	2		3	3	5
4	6	3	3		2	3	5
6	8	3		2	2	5	2
8	10	2	3	1		3	3
10	12	4	2	3	1	7	3
12	14	4		2		6	
14	16	2		3		5	
16	18	5		1		6	
18	20	3	1	3		6	1
20	22	1			1	1	1
22	24	3				3	
24	26	1				1	
26	Above	1				1	
Total		35	18	15	14	50	32
Ave. Speed Change		13.51	5.33	13.40	5.00	13.48	5.18
Std. Dev.		6.73	5.05	4.09	5.50	6.02	5.17



TABLE A23
FREQUENCY OF
RATIOS OF SPEED CHANGES TO APPROACH SPEEDS FOR INDIVIDUAL
VEHICLES DURING TRANSITION FROM 4-LANES TO 2-LANES
ON U. S. 52 NEAR TEMPLETON, INDIANA

$\frac{v_0 - v_1}{v_0}$		Not Meeting		Meeting		Total	
		P	T	P	T	P	T
.00	.02		1		1		2
.02	.04	1	6		4	1	10
.04	.06	2	2		2	2	4
.06	.08	2			1	2	1
.08	.10	1	1		1	1	2
.10	.12	2	1	1	2	3	3
.12	.14	3	1			3	1
.14	.16	1	1	1	1	2	2
.16	.18		2	2		2	2
.18	.20	2	1	2		4	1
.20	.22	4		1	1	5	1
.22	.24	1	1	2		3	1
.24	.26	3		2		5	
.26	.28	2		1		3	
.28	.30	2		1		3	
.30	.32	2	1	1	1	3	2
.32	.34	3		1		4	
.34	Above	4				4	
Total		35	18	15	14	50	32
Ave. Ratio		.213	.102	.223	.091	.216	.098
Std. Dev.		.101	.086	.062	.084	.091	.084

TABLE A24
 FREQUENCY OF
 e^{-F_0} FACTORS FOR INDIVIDUAL VEHICLES DURING TRANSITION
 FROM 4-LANES TO 2-LANES ON U. S. 52 NEAR TEMPLETON, INDIANA

e^{-F_0}		Opposing Lane				Total	
		Free		Occupied			
		P	T	P	T	P	T
.00	.01	1	7		5	1	12
.01	.02	3	2		3	3	5
.02	.03	4	2		2	4	4
.03	.04	4	1	2	1	6	2
.04	.05	1	3	2	1	3	4
.05	.06	4	1	3	1	7	2
.06	.07	3	1	2		5	1
.07	.08	4		2		6	
.08	.09	2		2		4	
.09	.10	3	1	2	1	5	2
.10	.11	2				2	
.11	.12	1				1	
.12	.13	1				1	
.13	.14						
.14	.15	1				1	
.15	.16	1				1	
Total		35	18	15	14	50	32
Ave. e^{-F_0}		.0644	.0277	.0643	.0250	.0644	.0266
Std. Dev.		.0382	.0261	.0202	.0257	.0336	.0255

TABLE A25
FREQUENCY OF
F_o VALUES FOR INDIVIDUAL VEHICLES DURING TRANSITION
FROM 4-LANES TO 2-LANES ON U. S. 52 NEAR TEMPLETON, INDIANA

F _o		Opposing Lane					
		Free		Occupied		Total	
		P	T	P	T	P	T
1.5	2.0	2				2	
2.0	2.5	9	1	3	1	12	2
2.5	3.0	12	2	8	1	20	3
3.0	3.5	4	4	4	2	8	6
3.5	4.0	4	2		3	4	5
4.0	4.5	3	2		1	3	3
4.5	5.0	1	1		3	1	4
5.0	5.5		5		2		7
5.5	6.0		1		1		2
Total		35	18	15	14	50	32
Ave. F _o		2.92	4.08	2.18	4.11	2.88	4.09
Std. Dev.		0.75	1.08	0.35	1.03	0.65	1.04

TABLE A26

VEHICLE SPEEDS ON RAMP AND MERGING SECTION
OF APPROACH TO WM. HARRISON BRIDGE
IN WEST LAFAYETTE, INDIANA

Sta.	Vehicle Speed (miles per hour)	
	P (42)	T (5)
-4+75	28.70	26.49
-4+25	29.89	28.03
-3+75	30.85	28.16
-3+25	31.36	28.94
-2+75	31.88	28.92
-2+25	32.22	29.14
-1+75	31.66	29.67
-1+25	30.69	29.66
-0+75	29.74	29.36
-0+25	28.13	28.08
0+25	26.85	28.26
0+75	25.64	28.21
1+25	25.24	28.81
1+75	25.32	29.30
2+25	27.13	29.98
2+75	28.85	30.56
3+25	30.44	30.85
3+75	32.36	31.36

TABLE A27

FREQUENCY DISTRIBUTION OF FACTORS (v_o , v_l , $v_o - v_l$, $\frac{v_o - v_l}{v_o}$, e^{-F_o} , F_o)

FOR MERGING SECTION OF APPROACH TO WM. HARRISON BRIDGE IN
WEST LAFAYETTE, INDIANA

v_o mph		Frequency P	v_l mph		Frequency P	$v_o - v_l$ mph		Frequency P
24	25	1	10	12	3	Below -4		2
25	26	1	12	14	2	-4 -2		4
26	27	2	14	16	1	-2 0		2
27	28	4	16	18	1	0 2		6
28	29	2	18	20	2	2 4		6
29	30	2	20	22	4	4 6		1
30	31	5	22	24	3	6 8		4
31	32	4	24	26	6	8 10		3
32	33	2	26	28	5	10 12		5
33	34	6	28	30	3	12 14		1
34	35	6	30	32	5	14 16		2
35	36	2	32	34	2	16 18		2
36	37	2	34	36	2	18 20		2
37	38	1	36	38	2	20 22		2
Above	38	2	Above	38	1			
Total		42			42			42

TABLE A27 (continued)

$\frac{v_0 - v_1}{v_0}$	Frequency P	$e^{-F_{0*}}$	Frequency P	F_{0*}	Frequency P
Below -0.15	1	-0.05	-0.04	1	1.0 1.5 3
-0.15 -0.10	2	-0.04	-0.03	1	1.5 2.0 5
-0.10 -0.05	3	-0.03	-0.02	1	2.0 2.5 6
-0.05 0.00	2	-0.02	-0.01	4	2.5 3.0 7
0.00 0.05	6	-0.01	0.00	1	3.0 3.5 3
0.05 0.10	3	0.00	0.01	5	3.5 4.0 2
0.10 0.15	3	0.01	0.02	4	4.0 4.5 3
0.15 0.20	2	0.02	0.03	1	4.5 5.0 2
0.20 0.25	3	0.03	0.04	2	5.0 5.5 2
0.25 0.30	3	0.04	0.05	1	5.5 6.0 1
0.30 0.35	1	0.05	0.06	1	
0.35 0.40	3	0.06	0.07	3	Discontinuous
0.40 0.45	4	0.07	0.08	3	-5.0 -4.5 2
0.45 0.50	1	0.08	0.09		-4.5 -4.0 3
0.50 0.55	1	0.09	0.10	1	-4.0 -3.5 1
0.55 0.60		0.10	0.11	2	-3.5 -3.0 1
0.60 0.65	3	0.11	0.12	1	-3.0 -2.5 1
0.65 0.70	1	0.12	0.13	1	
		0.13	0.14	2	
		0.14	0.15	2	
		Above 0.15	5		
Total	42		42		42

* Calculated by equation 12.

TABLE A28

VEHICLE SPEEDS BY STATIONS ON THE APPROACH
TO THE NARROW BRIDGE ON INDIANA 43
NEAR CHALMERS, INDIANA

Sta.	Speed (mile per hour)	
	P (39)	T (13)
-8+25	58.95	51.00
-7+75	59.14	50.87
-7+25	58.85	50.68
-6+75	59.11	50.74
-6+25	59.07	50.60
-5+75	50.17	50.62
-5+25	59.37	50.72
-4+75	59.44	50.95
-4+25	59.52	50.48
-3+75	59.01	50.37
-3+25	59.15	50.64
-2+75	59.27	50.60
-2+25	59.21	50.46
-1+75	59.24	50.54
-1+25	58.70	50.02
-0+75	58.34	49.74
-0+25	57.92	50.02

VITA

VITA

Kenneth John Tharp was born at Oskaloosa, Iowa on February 23, 1924. He attended Oskaloosa High School and graduated in 1942. He enrolled at Iowa State University that same year.

From August 1943 until October 1945 he served in the Army of the United States and saw combat service in the European Theater of Operations. Following his honorable discharge, he returned to Iowa State University and completed the requirements for the degree of Bachelor of Science in Civil Engineering in December 1947.

After graduation, Mr. Tharp worked for consulting engineering firms and municipalities prior to accepting a teaching position on the Civil Engineering staff at Duke University, Durham, North Carolina. While at Duke University (1953-1960) he operated a small consulting engineering business. Also he enrolled at North Carolina State College as a candidate for a Master of Science degree in Civil Engineering. This degree was awarded in June 1961.

Mr. Tharp is married and the father of three children. He is a member of the Lutheran Church in America and has been active in local congregations.

He is a registered Professional Engineer (civil and structural branches) and Land Surveyor in the State of Iowa. He is a member of the American Society of Engineering Education and Phi Kappa Phi honorary society.

The thesis for his Master of Science degree was entitled "The Future Mass Transportation System for the City of Durham, North Carolina." Other publications are:

Tharp, K. J., "Truck Weight-Speed Study," Joint Highway Research Project, Purdue University, Lafayette, Indiana, 1961.

Tharp, K. J. and D. F. Petty, Traffic Speed Report No. 74, Joint Highway Research Project, Purdue University, Lafayette, Indiana, 1961.

